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Asphalt mixtures improved with plastic additives: mix design and case study in an airport

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"The principle of science, the definition, almost, is the following: The test of all knowledge is experiment. Experiment is the sole judge of scientific truth".

(Richard Feynman, Lectures on Physics, 1963)

"L'università deve essere focolaio di attività scientifica, vero laboratorio nel quale maestri e scolari collaborano ad indagare nuovi veri e a rivedere questioni già discusse. Così nello studente si educa lo spirito critico e, quel che più importa dato lo scopo speciale che la nostra università ha, lo spirito di ricerca".

(Agostino Gemelli, 1919)

"If politics is the art of the possible, research is surely the art of the soluble. Both are immensely practical-minded affairs".

(Sir Peter B. Medawar, The Art of the Soluble, 1969)

To my family,
to my fiancée
and
to my grandfather Matteo,
who from up there is my first fan in this important achievement!

Acknowledgements

When you complete a long journey you cannot think of having done it all by yourself.

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Special thanks to my supervisor Prof. Clara Celauro for her assistance and guidance throughout my research; to Prof. Joel R. M. Oliveira and his team for their hospitality and helpfulness during my research period at the Laboratory of Road Materials of the University of Guimarães (Portugal); to Denis Gailor for helping with the English revision of the dissertation; to the Iterchimica S.r.l. and I.s.a.p. S.r.l. companies for the materials supplied, needed for my research work and finally I want to thank Antonino Lorello for his support with learning the basic procedures for conducting various tests.

Abstract

Today the growing demand for performance and the need to protect the ecosystem lead engineers to develop new manufacturing technologies and to experiment the use of new materials. The criterion followed is to promote more rational use of available resources and low environmental impact techniques.

A method that can help improve the quality of pavements, avoiding degradation, with benefit and savings regarding the need for maintenance, is the addition of polymers in asphalt mixtures.

The fact is that the use of polymers makes it possible to increase the performance of asphalt mixtures, decreasing production and laying costs, and reducing the environmental impact. Not only that, but for sustainable development it makes it possible to reduce consumption of valuable natural resources.

The aim of this study is the optimization of mixtures that do not necessarily employ high-performance materials, but make use of locally available stone aggregate and bitumen, aiming to improve the traditional mixtures, as is made possible by addition of polymers.

For this purpose, the mixtures studied were subjected to various laboratory tests, such as the Marshall test, the gyratory compactor test, water sensitivity test, wheel-tracking test, four-point bending tests and triaxial cell tests and field tests like the HWD test.

The test results showed the advantage of using polymers, and especially waste, in the asphalt mixtures in technical, economic and environmental terms, and thus justify their use in settings where the pavements are subject to high loads, which lead to gradual surface degradation.

KEYWORDS: Asphalt modification, Polymers, Plastics, Wet process, Dry process, EME, Mixture performance, Airport pavement, Bakfaa.

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Introduction

Today the growing demand for performance and the need to protect the ecosystem lead engineers/designers to develop new manufacturing and monitoring technologies, and to experiment the use of new materials, and to improve analysis models as well as design methods of pavements and mixtures. The criterion followed is to promote more rational use of available resources and low environmental impact techniques (Losa & Dringoli, 2007).

Therefore the demand for superior performance compared to that of traditional asphalt mixtures, the recovery and recycling of existing pavements, the use of new materials combined with new production technologies have given rise to so-called special pavements. These have particular characteristics regarding particle size fractions and high quality materials. This category includes, for example, asphalt mixtures with additives, high modulus asphalt mixtures, also called Enrobés à Module Élevé (EME), split mastix asphalt (SMA), thin asphalt overlays, open graded drainage and sound absorption asphalt mixtures, rubber-asphalt mixtures, low-energy asphalt mixtures, also called warm mix asphalt (WMA), and asphalt mixtures with reclaimed asphalt pavement (RAP).

The incorporation of waste materials and other industrial products as construction materials is also discussed, as being one of the main solutions used by practitioners to respond to society's current sustainability issues.

In particular, the introduction of plastic waste in asphalt binders and mixtures is an excellent alternative to landfill disposal (Celauro et al. 2006), because asphalt modification by polymer incorporation can significantly improve the performance of road pavements (Bense, 1983;

Serfass et al., 2000; Rahi et al., 2015), if a rigorous and appropriate selection is made in terms of plastic waste and production conditions (Abreu et al., 2015).

And so, for economic and technical reasons, what we are currently witnessing is a phenomenon of expansion on the market of additives used in the technology of asphalt mixtures. These products are used in order to improve performance and reduce the costs for asphalt pavement production and exploitation. In general, meeting all these goals with a single product is difficult, because higher quality requires higher costs. The latest researches in this area have resulted in products whose quality increases performance at minimal cost (Iliescu & Pop, 2010).

Despite legislative initiatives aimed at reducing the use of polymers, the polymer market shows a strong potential for development and diversification. The reasons of this success are clear: low weight, workability, versatility, hygiene, different selection options, recycling and recovery (Giuffré & Di Francisca, 2001).

Although it has been known for some time that virgin polymers can improve modified bitumen's properties, nowadays there are some concerns about replacing virgin materials with recycled polymers (Gonzalez et al., 2002).

My research activity carried out during the Doctorate course must be considered within this scenario.

The primary objective of this study was to assess technological feasibility, in terms of mix design and production and application of the asphalt mixtures with polymeric additives. Therefore to achieve the main aim of the study, the following tasks were performed:

- Task 1 was to survey published literature regarding implementation and practice of the mixtures with additives. This extensive literature review includes national studies, as well as other available reports and articles from European countries and the world as a whole (literature review).
- Task 2 was to fabricate specimens and to perform various key laboratory tests in order to identify mixture properties and performance characteristics (experimental design and materials selection, laboratory testing program).
- Task 3 was to analyse the laboratory test results and to use the test data for a comparison with field performance data (laboratory test data analysis and comparison).

Thesis layout is reported in Fig. I.1. This dissertation is composed of four chapters. As a first step to achieving the project objectives, an extensive literature review was conducted to present background information associated with the benefits and approaches regarding the use of polymeric additives in asphalt mixtures as well as using waste materials (Chapter 1). Chapter 1 is also an introductory chapter outlining the problem statement.

The cases studied are presented in subsequent chapters. Case 1 regards asphalt surface mixtures with improved performance using waste polymers via dry and wet processes and the tests were performed at the Road Materials Laboratory of the University of Guimarães (Portugal). This experimental study led to important conclusions from a comparison between two different processes (WET and DRY) used for production of modified asphalt mixtures with waste polymers. Moreover the evaluation of the effects of polymer types used is presented (Chapter 2).

Case 2 is an experimental study concerning the development and optimization (mix design) of special asphalt mixtures for binder and base courses, also with additives specifically engineered. Specifically, the focus was on the optimization of the high modulus asphalt mixtures making use of ordinary aggregate and bitumen (as locally available) instead of the very hard bitumen typically prescribed, also aiming to improve the traditional mixture as made possible by suitable polymers (Chapter 3).

Case 3 is an experimental study concerning plastic additives used in the surface, binder and base courses of the access pavement to the new apron of Palermo International Airport. The focus was on comparison between data obtained from laboratory tests and data obtained from field tests made it possible using BAKFAA, a software program for back-calculation analysis made available by the Federal Aviation Administration (FAA), in which deflection values obtained by Heavy Weight Deflectometer (HWD) tests were used (Chapter 4).

Finally a list of conclusions as results of the research work as well as recommendations for future work are presented.

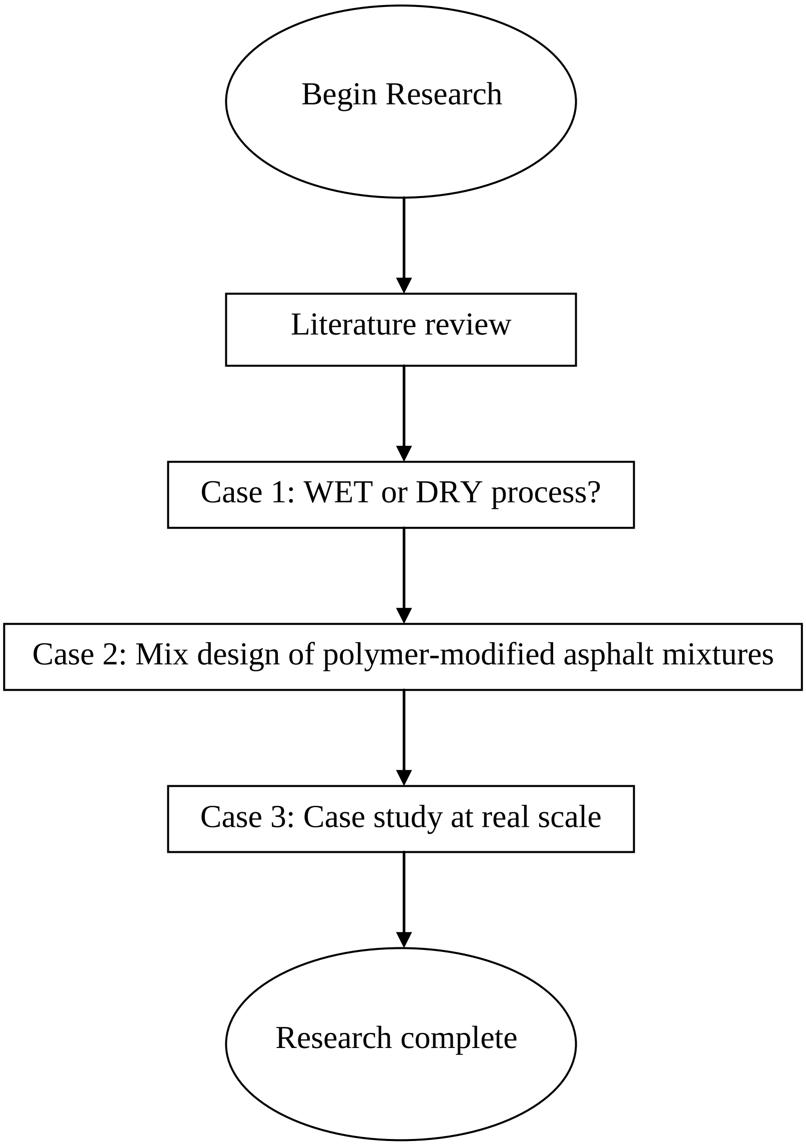

Fig. I.1 Thesis layout

Chapter 1

Background and literature review

1.1 General

The state of a road pavement is characterized by functional (adherence, regularity, noise) and structural (bearing capacity) properties, which together combine in order to make vehicular traffic permanently possible in terms of safety, comfort and economy. When one or more of these related properties are lacking, not only is the road surface inadequate to allow traffic circulation in compliance with the required standards (lack of adherence = slippery road; irregular viable plan = unsafe and uncomfortable driving) but there are also the conditions for the deterioration of the pavement service life, these properties being related to each other: road irregularities lead to a dynamic load increase, which affects the bearing capacity over time; the structural failures cause unevenness, depressions, potholes, and changes in cross-slope with reductions in driving safety. For this reason, through an appropriate design of the road surface (or careful planning of management in already existing pavements) the implementation of high performance pavements was undertaken using appropriate materials and thicknesses which could represent a durable and reliable solution to increased vehicular traffic (Pasetto, 1998).

Increased heavy vehicle traffic, geometric and weight changes in load transfer systems on the road pavement, together with the widespread tendency to travel overloaded, have created problems in most existing road pavements (Giuffré & Di Francisca, 2001).

Over the years the peculiar characteristics of flexible pavements have remained unchanged; however, this did not exclude looking for progress and introduction of new elements. Today the growing demand for performance and the need to protect the ecosystem lead engineers/designers to develop new manufacturing and monitoring technologies, and to experiment the use of new materials, and to improve analysis models as well as design methods of pavements and mixtures. The criterion followed is to promote more rational use of available resources and low environmental impact techniques (Losa & Dringoli, 2007).

For economic and technical reasons, we are currently witnessing a phenomenon of expansion on the market of additives used in the technology of asphalt mixtures. These products are used in order to improve performance and reduce the costs for asphalt pavement production and exploitation. In general, meeting all these goals with a single product is difficult, because higher quality requires higher costs. The latest researches in this area have resulted in products whose quality increases performance at minimal cost (Iliescu & Pop, 2010).

Several factors influence the performance of flexible courses, e.g., the properties of the components (binder, aggregate and additive) and the proportion of these components in the mix (Awwad & Shbeeb, 2007).

Bitumen is the only deformable element of a pavement and has a very important role in pavement performance (Becker et al., 2001).

At high temperatures (40 to 60°C), asphalt exhibits viscoelastic behaviour. Pavements made of asphalt may show distress when exposed to high temperatures. At elevated temperatures, permanent deformation (rutting) occurs and leads to channels in the direction of travel. This is attributed to the viscous flow of the asphalt matrix in paving mixtures, which retains strains induced by traffic. Therefore, pavement performance is strongly associated with the rheological properties of asphalt cement. Increased traffic factors such as heavier loads, higher traffic volume, and higher tyre pressure demand higher performance pavements. A high performance pavement requires asphalt cement that is less susceptible to high temperature rutting or low temperature cracking and has excellent adherence to stone aggregates (Chen et al., 2002).

Some improvements in asphalt properties have been gained by selecting the proper starting crude, or tailoring the refinery processes used to make asphalt. Unfortunately, there are only a few crudes that can produce very good asphalts, and only a limited number of actions that can be taken to control the refining process to make improved asphalts (Becker et al., 2001). The next step taken by the industry was to modify the asphalt. Air blowing makes asphalt harder (Aflaki & Tabatabaee, 2009; Corte, 2001; Lee et al., 2007). Fluxing agents or diluent oils are sometimes used to soften the asphalt. Another method that can significantly improve asphalt quality is the addition of polymers (Becker et al., 2001).

1.2 Innovative asphalt mixtures

The demand for superior performance compared to those of traditional asphalt mixtures, the recovery and recycling of existing pavements, the use of new materials combined with new production technologies have given rise to so-called special pavements. These have particular characteristics regarding particle size fractions and high quality materials. This category includes, for example, asphalt mixtures with additives, high modulus asphalt mixtures, also called Enrobés à Module Élevé (EME), split mastix asphalt (SMA), thin asphalt overlays, open graded drainage and sound absorption asphalt mixtures, rubber-asphalt mixtures, low-energy asphalt mixtures, also called warm mix asphalt (WMA), and asphalt mixtures with reclaimed asphalt pavement (RAP). The next sections will focus only on the first two asphalt mixtures.

The incorporation of waste materials and other industrial products as construction materials is also discussed, as being one of the main solutions used by practitioners to respond to society's current sustainability issues.

In the road sector, the most widely studied wastes are reclaimed asphalt material (Dinis-Almeida et al., 2016; Silva et al., 2012) and construction and demolition waste (Bestgen et al., 2016; Gómez-Meijide et al., 2016; Pasandín et al., 2015; Sangiorgi et al., 2015; Dondi et al., 2014), but other wastes have also been investigated, e.g., plastic waste (Lastra-González et al., 2016; García-Travé et al., 2016; Karmakar & Roy, 2016; Modarres & Hamedi, 2014) or waste tyre rubber (Oliveira et al., 2013; Wang et al., 2015; Xu et al., 2015; Rahman et al., 2010; Airey et al., 2004; Rahman et al., 2004).

In particular, the introduction of plastic waste in asphalt binders and mixtures is an excellent alternative to landfill disposal (Celauro et al., 2006), because asphalt modification by polymer incorporation can significantly improve the performance of road pavements (Bense, 1983; Serfass et al., 2000; Rahi et al., 2015).

Several field studies (Raad et al., 1997; Zubeck et al., 2002; Ponniah & Kennepohl, 1995; Foster & Hein, 1999; Wegman et al., 1999; Von Quintus & Killingsworth, 1998) have shown that the performance enhancement is not uniform and varies as a function of the site, design, materials and construction factors.

Table 1.1 is a generic classification system regarding additives in HMA mixtures.

Type		Generic
1.	Filler	Mineral filler (Crusher fines, Lime, Portland cement, Fly ash), Carbon black
2.	Extender or chemical modifiers	Sulphur, Lignin and certain organo-metallic compounds
3.	Polymers	Thermoplastic elastomers [Rubber (natural rubber), Natural latex (Styrene-butadiene or styrene-butadiene rubber), Synthetic latex (Polychloroprene latex), Block copolymer (Styrene-butadiene-styrene SBS, Styrene-isoprene-styrene SIS), Reclaimed rubber (Crumb rubber modifier)], Thermoplastic polymers or plastics (Polyethylene/polypropylene, Ethylene acrylate copolymer, Ethylene-vinyl acetate EVA, Polyvinyl chloride PVC, Ethylene-propylene or ethylene-propylene diene monomer, Polyolefin), Thermosetting polymers (resins: epoxy resin, acrylic resin, polyurethane resin, phenolic resin), Combinations (Blends of the above)
4.	Fibre	Natural (Asbestos, Rock wool), Manufactured (Polypropylene, Polyester, Fibreglass, Mineral, Cellulose)
5.	Oxidant	Manganese salts
6.	Antioxidant	Lead compounds, Carbon, Calcium salts
7.	Hydrocarbons	Recycling and rejuvenating oils, Hard and natural asphalt (Gilsonite, Trinidad lake asphalt, rock asphalt)
8.	Antistripping agents	Amines (liquid antistripping additives), Lime
9.	Waste materials	Roofing shingles, Recycled tyres, Glass
10.	Miscellaneous	Silicones, Deicing calcium chloride granules

Tab. 1.1 Generic classification of asphalt additives (Roberts et al., 1996, Nikolaides, 2014)

1.3 Asphalt mixtures with polymeric additives

A method that can help improve the quality of road pavements is the addition of polymers (Bense, 1983; Serfass et al., 2000). Some polymer systems are being increasingly used in asphalt concrete pavements because of their role in reducing several types of pavement distress (fatigue cracking, rutting, temperature cracking and stripping) and enhancing pavement performance (Goodrich, 1988; Little et al., 1987).

The term ‘‘ polymer ’’ simply refers to very large molecules made by a chemical reaction (polymerization) of many small molecules (monomers) in order to produce long chains. The physical properties of a specific polymer are determined by the sequence and chemical structure of the monomers from which it is made, its molecular weight and molecular weight distribution (Becker et al., 2001).

Despite legislative initiatives aimed at reducing the use of polymers, the polymer market shows a strong potential for development and diversification. The reasons of this success are clear: low weight, workability, versatility, hygiene, different selection options, recycling and recovery (Giuffré & Di Francisca, 2001).

These qualities have led, first slowly and then starting from the 1930s, with increasing intensity to the study of new polymers and the development of polymer science, which still plays a central role today in pure and applied scientific research by many universities and sector companies (Zhu et al., 2014).

Polymers can be classified in many ways, but most commonly they are classified by their physical properties. They may also be classified according to their chemical sources, but, depending on their physical properties, they may be classified as thermoplastic and thermosetting materials. Thermoplastic materials can be formed into desired shapes under heat and pressure and become solids on cooling. If they are subjected to the same conditions of heat and pressure, they can be remoulded. Thermosetting materials, once shaped, cannot be softened/remoulded by the application of heat.

Some typical thermoplastic materials are polyethylene teryphthalate (PET), polypropylene (PP), polyvinyl acetate (PVA), polyvinyl chloride (PVC), polystyrene (PS), low density polyethylene (LDPE) and high density polyethylene (HDPE). Thermosetting materials are bakelite, epoxy, melamine, polyester, polyurethane, urea-formaldehyde and alkyd (CPCB 2009).

The most commonly used asphalt mixtures modifiers are thermoplastic polymers. In the domain of thermoplastic polymers there are two families that differ, mainly, by characteristics related to stiffness, elasticity, deformability: elastomers and plastomers.

Elastomers require a supply of mechanical energy in order to be mixed with the bitumen. For this reason, they are mixed with bitumen before the mixture is prepared. Modified bitumens with elastomer are usually obtained by strong mechanical mixing at a higher temperature than the flow temperature of the polymer.

Plastomers do not require additional power for mixing, thus they can be previously mixed with bitumen or they can be introduced directly into the mixer. The solution of introducing the modifier directly into the mixer can be adopted if only plastomers are used. In the case of plastomers with elastomer association, the bitumen will be previously modified.

Regarding elastomers, they raise some issues related to the following:
- compatibility with bitumen;
- storage stability;
- high stability, which can cause problems during manufacturing and application of the mixtures.

The benefits of using elastomers are the following:
- considerable reduction of thermal susceptibility;
- increased flexibility at low temperatures;
- increased stiffness at high temperatures.

The disadvantages of the use of elastomers are the following:
- increased viscosity at high temperatures;
- limited stability at storage;
- additional energy consumption for transport, storage and application.

The most commonly used plastomers in road works are the following: EVA (ethylene-vinyl acetate), EMA (ethylene-methyl acrylate) and EBA (ethylene-butyl acrylate). Their structure is composed of a hydrocarbon skeleton which provides rigidity and cohesion, including the crystalline fractions which regulate thermal susceptibility, on which the polar comonomer is fixed, making it possible to control the compatibility of adhesiveness and crystallinity.
The benefits of using plastomers are the following:
- decreased thermal susceptibility;
- increased stiffness at high temperatures.

The disadvantage of the use of plastomers is the following:
- fragility at low temperatures.

In order to eliminate the disadvantage related to the fragility of plastomers at low temperatures, the ideal solution is to have a higher dosage of bitumen and to associate the plastomers with fibres.

Although the addition of virgin polymers is in accordance with the purpose of improving the properties of the asphalt mixtures, the use of recycled polymers can also show similar performance (Ahmadinia et al., 2012: Ahmadinia et al., 2011), if a rigorous and appropriate selection is made in terms of plastic waste and production conditions (Abreu et al., 2015).

The use of waste thermosetting polymers (12 million tons of waste polymers are presently dumped into landfills, every year, in Europe) can be considered as a sustainable technology, given that an equivalent performance can be assured (Silva et al., 2011; Bense, 1983; Maze et al., 2000; Serfass, 2000). In fact, the aim of the introduction of waste polymers in asphalt binders and mixtures is not to be an alternative to landfill, but to improve the performance of mixtures (Celauro et al., 2006), if these are properly designed.

Many studies have shown the use of polymers (Awwad & Shbeeb, 2007; Chen et al., 2002) and waste polymers (Gawande et al., 2012; Kalantar et al. 2012; Celauro et al., 2001a; Celauro et al., 2001b; Celauro, 2005; Zoorob & Suparma, 2000; Sabina et al., 2009; Naskar et al., 2010; Garcia-Morales et al., 2005) as additives for asphalt mixtures.

Polymers can be used in different ways: as an additive they may be added directly during the mixing process or they can be mixed with bitumen to improve the rheological properties or, in the case of high melting temperature plastics, as a substitute for particle size fractions.

Asphalt mixtures with plastics (virgin or waste) are called additivated asphalt mixtures, to emphasize the fact that there are interactions between additive and asphalt mixtures, unlike what occurs instead in modified bitumens, in which both mechanical and chemical interactions between a certain fraction of the bitumen (maltens) and the polymers occur (Giuffré & Di Francisca, 2001).

There are two main processes to add polymers to asphalt mixtures, namely by modifying the bitumen (WET process) or by adding the polymers during the mixing phase (DRY process) (Pettinari et al., 2014). However, the WET process needs specific equipment (for mixing and to facilitate the reaction with bitumen at high temperatures), while this is not required for the DRY process. Therefore, even though the DRY process is easier to implement, the WET process has the advantage of controlling the properties of the binder (Celauro et al., 2004), and that is the reason why bitumen modification is the most widely used process.

The solution of modifying bitumen with polymers in order to obtain polymer-modified bitumen was previously far more widespread because of the potential for analysis of the properties of bitumen before use. Modifying bitumen in the mixture solution is newer and it is much easier because complications associated with transport and storage of modified bitumen and also with the high energy consumption for its production are eliminated. In this case, the modifier is in the form of granules, which are inserted directly into the mixture, without changing the manufacturing process very much (Iliescu & Pop, 2010).

1.3.1 Wet process

The limited oil resources for producing good-quality bitumen and the lack of effective control actions during refinement, as well as the driving force of earning the maximum economic benefits, made industries pay more attention to bitumen modification (Becker et al., 2001). Following the rapid development, increased traffic load, higher traffic volume and insufficient maintenance led to many severe distresses (e.g. rutting and cracking) of road surfaces. The harsh reality was demanding more of bitumen quality. In order to obtain bitumen with enhanced quality, an increasing number of investigations also began to focus on bitumen modification. Among all attempted or investigated bitumen modification methods, polymer modification has been one of the most popular approaches.

Polymer modification of bitumen is the incorporation of polymers in bitumen by mechanical mixing or chemical reaction (Lu, 1997). The various polymers investigated have made it possible to improve some properties of bitumen, such as higher stiffness at high temperatures, higher cracking resistance at low temperatures and better moisture resistance or longer fatigue life (Tayfur et al., 2007; Isacsson & Zeng, 1998; Gorkem & Sengoz, 2009; Alataş & Yilmaz, 2013, Ponniah & Kennepohl, 1996; Von Quintus et al., 2007). Effective polymer modification results in a thermodynamically unstable but kinetically stable system in which the polymer is partially swollen by the light components of bitumen (Polacco et al., 2006). Some important factors, including the characteristics of the bitumen and the polymer themselves, the polymer content and the manufacturing processes, determine the final properties of polymer-modified bitumen (PMB) (Lu, 1997; Larsen et al., 2009). As the polymer content increases, phase inversion may occur in some PMBs: from bitumen being the dominant phase to polymer becoming the dominant phase (Sengoz & Isikyakar, 2008). However, an ideal microstructure for PMB contains two interlocked continuous phases, which determines the optimum polymer content for bitumen modification (Brûlé et al., 1988).

Since a modified binder consists of two distinct phases, three different cases must be considered (Brûlé, 1996):

- low polymer content;
- polymer content around 5%;

- sufficiently high polymer content.

In the first case, the bitumen is the continuous phase of the system, and the polymer phase (polymer content less than 4%) is dispersed through it. Due to its lowered oil content, the bitumen phase has a correlatively higher asphaltene proportion. The dispersed polymer phase enhances the properties of the binder both at low and at high service temperatures. In other words, the polymer extends the useful temperature range for the asphalt. In this case, the choice of bitumen is a determining factor. These materials are usually employed for paving.

In the second case the system may show microstructures in which the two phases are continuous and interlocked. Such systems are generally difficult to control and pose stability problems.

In the last case (polymer content more than 7%) the polymer phase is the matrix of the system. This is in fact not a bitumen, but a polymer plasticized by the oils in the bitumen in which the heavier fractions of the initial asphalt cement are dispersed. In this case, the polymer is the continuous phase and the asphalt is dispersed in it. The properties of such a system are fundamentally different to those of bitumen and depend essentially on those of the polymer. One should speak not of a polymer modified bitumen, but of a thermoplastic adhesive. These materials are usually employed for roofing (Becker et al., 2001).

The polymer content ranges between 2% and 10% by weight of the bitumen. In the previous decade the most common proportions were about 5% or 6% but within the last few years a lower polymer content (2-3%) has been preferred (Kalantar et al., 2012)

The best results have been obtained when polymer concentration was kept below 3% (Habib et al., 2010).

A three-year study was conducted at Michigan State University to characterize polymer-modified asphalt mixtures. It was found that the rheological and engineering properties of these mixtures largely depend on the polymer type and content (Khattak & Baladi, 2001).

The United States, China, France and Italy are leaders in polymer-modified asphalt research and development activities, even though considerable work has also been done in Japan, Germany, Russia, Great Britain, and Canada (Becker et al., 2001).

As mentioned before, the polymer must be compatible with the bitumen and maintain this compatibility during storage and use. This is a difficult task, because of the big difference in molecular weight and structure, viscosity and density of constituents (Giavarini et al., 1996).

The general conclusion from the studies on the nature of the asphalt says that to dissolve and expand the polymer asphalt should contain enough oil fractions. It should also have a high content of condensed ingredients like aromatic hydrocarbons which mix especially well with polar aromatic polymers (Zielinski et al., 1995).

Generally, a thermoplastic polymer-modified asphalt which results from physical mixing of the constituents without chemical interactions can consequently be a two-phase system. One phase is a swollen polymer and another phase grouping the constituents of the asphalt not intervening in the solvation.

Vonk and Bull have shown that elastomer of a thermoplastic rubber copolymer can absorb almost all the bitumen components except the asphaltenes (Vonk & Bull, 1989). Therefore the asphaltene content of the bitumen should not be too high; otherwise addition of a thermoplastic rubber can result in asphaltene precipitation or gelation and will result in phase separation so the blend becomes unworkable. On the other hand, if the asphaltene content is low a single phase blend may be obtained.

The mixing process is influenced by a number of parameters:

- Nature of the polymer.
  The proper mixing time to achieve a homogeneous blend of the bitumen and polymer depends on the type, molecular weight and chemical composition of polymer. A polymer with higher molecular weight needs a longer time to blend with bitumen and vice versa (Morgan & Mulder, 1995).
- Physical form of the polymer.
  A smaller particle size has a larger surface area per unit mass of polymer. Thus, swelling of the polymer and penetration are easier (Morgan & Mulder, 1995).
- Type of mixing equipment.
  There are two main methods for mixing bitumen with polymer, namely, high shear and low shear mixing. The low shear mixer is a simple mixing tank with a paddle stirrer. It can be used to mix bitumen with a powdered modifier. The mixing process is limited to swelling and dissolving the bitumen with polymer. The temperature is fixed during the mixing. The high-shear mixer reduces the polymer particles size by mechanical and hydrodynamic shear. The

temperature will increase during the mixing in order to dissolve polymer into the bitumen and make a homogeneous blend.
- Time-temperature profile during mixing.
  The structure and properties of PMA are a function of the blending conditions. It means the longer the mixing time is, the finer the microstructure will be and the higher the temperature is, the more rapidly this process will be done (Becker et al., 2001). The Shell report (Lu & Isacsson, 1997) suggests that the mixing temperature should not exceed 185°C. Otherwise the bitumen would burn. Moreover, the mixing time should be adequate for homogeneous dispersion of the waste plastic within the bitumen matrix.
- Compatibility and stability.
  A polymer may be incompatible, slightly compatible or compatible with bitumen. In the first case, the mixture is heterogeneous in that the polymer affects the chemical equilibrium of the bitumen so that the mixture does not have enough cohesion and ductility (Lesueur, 2009; Brûlé & Druon, 1975; Kraus, 1982; Bouldin et al., 1990). Slightly compatible polymers can improve bitumen properties but they require a high-shear mixer with a high temperature to mix with bitumen homogeneously. Compatible polymers make it possible to obtain a physically stable blend. This kind of polymers may or may not improve the physical properties of the bitumen. Compatibility between polymer and bitumen should be high enough to avoid phase separation in the bitumen and to achieve a proper pavement with good quality (Kalantar et al., 2012).

The main reasons to modify asphalts with polymers could be summarized as follows (Lewandowski, 1994):
- to obtain softer blends at low service temperatures and reduce cracking (Ali et al., 1994);
- to reach stiffer blends at high temperatures and reduce rutting (Ali et al., 1994);
- to reduce viscosity at layout temperatures;
- to increase the stability and the strength of mixtures;
- to improve the abrasion resistance of blends;
- to improve fatigue resistance of blends;
- to improve oxidation and aging resistance;
- to reduce the structural thickness of pavements;
- to reduce the maintenance costs of pavements.

Figure 1.1 compares the stiffness of a conventional asphalt binder to an ideal modified asphalt binder at different in-service temperatures. As mentioned above, at high temperatures polymer modification increases binder stiffness and elasticity, as a result of an increased storage modulus and a decreased phase angle. Both increasing the storage modulus and decreasing the phase angle improve rutting resistance of the pavement (Bahia & Anderson, 1995). Instead, at low temperatures, polymer modification lowers the creep stiffness of the asphalt, which improves thermal cracking resistance (Isacsson & Lu, 1999).

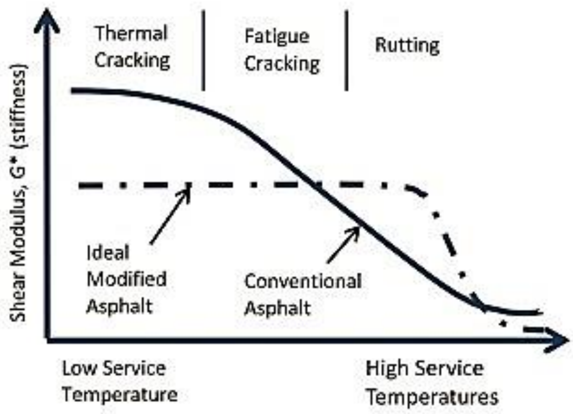

Fig. 1.1 Effects of polymer modification in asphalt binder (Epps, 1986)

Although there have been many experimental and field studies comparing the performance of polymer-modified asphalt (PMA) mixtures and conventional hot-mix asphalt mixtures, there has not been a concerted effort to quantify the benefits of using PMA mixtures or to develop guidance on when the use of PMA mixtures is cost-effective. It would be desirable to identify the site and design factors (e.g. traffic, climate, thickness, etc.) for which the effect of using PMA can be maximized.

For this purpose an investigation was conducted in North America. It was found that PMA mixtures significantly enhance not only the rutting performance of flexible pavements, but also their fatigue and fracture performance.

The life cycle of PMA mixtures has been found to be higher and this improvement varies from 2 to more than 10 years, which represents a 20% to 100% increase in performance (Von Quintus et al., 2007).

Polymer-modified binders have been used with success at high stress locations, such as intersections of busy streets, airports, heavy vehicle stations and race tracks (King et al., 1999).

Although these polymers all improve bitumen properties to some extent, there are still some drawbacks limiting the future development of bitumen polymer modification, such as high cost (the incorporation of polymers into asphalt increases the price of the product between 60 and 100%, Becker et al., 2001), low compatibility between polymer and bitumen, higher viscosity during the manufacturing process and application of the mixtures, low ageing resistance and poor storage stability of polymer modified bitumen (Zhu et al., 2014).

The poor storage stability of some PMBs usually results from poor compatibility between polymer modifiers and bitumen, which is controlled by polymers' and bitumen's different properties such as density, molecular weight, polarity, solubility (Wang et al., 2010) and also the chemical structure and reactivity of polymers (Chang et al., 2000).

At the same time, the addition of a polymer causes a significant increase in the production costs and adds operative complications that are mostly related to mixing and storage. With regard to the latter, the low compatibility between asphalt and polymer can lead to phase separation when the material is stored at a high temperature (160-200°C) in the absence of stirring. In such a case, a polymer-rich phase migrates to the higher part of the storage tank, while an asphalt-rich phase segregates into the lower part (Polacco et al., 2005).

Researchers have tried various solutions to remove the drawbacks of currently used polymer modifiers, among which saturation, functionalization (including application of reactive polymers) and using extra additives (sulphur, antioxidants and hydrophobic clay minerals). These solutions do overcome some disadvantages of PMB, but most cause some new problems (Zhu et al., 2014).

Although it has been known for some time that virgin polymers can improve modified bitumen's properties, nowadays there are some concerns about replacing virgin materials with recycled polymers (Gonzalez et al., 2002).

Since polymers are rather expensive, the amount of polymer used to improve the road pavement must be small. Recycled polymers have been found to show similar results in improving road performance as compared to virgin polymers. From the economic and environmental point of view using waste polymer as a modifier is beneficial because it can help to improve the performance of pavement and quality of the roads and also to solve the waste disposal problem (Gonzalez et al., 2002).

Using waste plastic bottles as a modifier in road surfaces can potentially help reduce material wastage and improve the performance of road surfaces at the same time (Huang et al., 2007; Sengoz & Topal, 2005; Xue et al., 2009; Arabani et al., 2010; Huang et al., 2009).

In this connection, the modification of bitumen with plastic wastes can improve the performance of asphalt mixtures in terms of rutting resistance, high temperature stiffness and susceptibility to temperature variations (Kim et al., 2013). Depending on the type of waste polymer used, better fatigue cracking resistance has also been found (Costa et al., 2013). The advantage of wet process is the following:

- it can be utilized for recycling of any type, size and shape of waste material (plastics, rubber, etc.).

The disadvantages of wet process are the following:

- it is time-consuming and requires more energy for blending;
- a powerful mechanical device is required;
- additional cooling is required as improper addition of bitumen may cause air pockets in roads;
- the maximum % of waste plastic that can be added around 8% (Gawande et al., 2012).

1.3.2 Dry process

Many studies have instead investigated the dry process (Awwas & Shbeeb, 2007; Giuffré & Di Francisca, 2001; Celauro, 2005; Celauro et al., 2001a; Celauro et al., 2001b; Celauro et al., 2006).

Regarding the timing of adding plastics, there are three possibilities: immediately after the aggregate and before the bitumen, or after the bitumen and before the filler or after the filler. Adding plastics immediately after the aggregate always makes it possible to obtain the best results. For this reason the best production process provides the following sequence of components in the mixing tank: stone aggregates, plastics, bitumen and finally filler. In this way not only the mechanical performance improves, but there is also a reduction of the risk of bitumen ageing, which can occur if the plastic is added after the bitumen.

Asphalt mixtures with plastics, unlike the case of modified bitumens, do not produce additional difficulties with regard to the workability and compactibility. Cleaning of the mixing equipment and the tools with which they are in contact does not pose particular problems, unlike in the case of modified bitumens (Giuffrè et al., 2001).
The advantages of the dry process are the following:

- the plastic is coated over stones, improving the surface properties of the aggregates;
- coating is easy and the required temperature is the same as road laying temperature;
- the use of more than 15% of waste plastic is possible;
- flexible films of all types of plastics can be used;
- it doubles the binding property of aggregates;
- no new equipment is required;
- bitumen bonding is stronger than normal;
- the coated aggregates show increased strength;
- replacing up to 15% of bitumen, higher cost efficiency is possible;
- no degradation of roads even 5-6 years after construction;
- it can be carried out in all types of climatic conditions;
- no evolution of any toxic gases, as the maximum temperature is 180ºC.

The disadvantage of the dry process is the following:

- the process is only applicable to plastic waste material (Gawande et al., 2012).

1.4 EME

Enrobé à Module Élevé (EME) or High-Modulus Asphalt (HiMA) is a French technology that was conceived in the 1980s with the purpose of maximizing stiffness and fatigue resistance, whilst ensuring that rutting and durability (particularly moisture resistance) requirements were still being met. It was initially intended to be used on the most heavily trafficked routes in France, as well as on airport pavements and container terminals, but early successes quickly opened up new avenues for its application. One of the fastest growing offsets of EME has been urban roads, based on the ability to reduce overall layer thickness as a result of the substantially higher stiffness of the material, while still being able to maintain the same level of performance. This has translated into direct savings in road construction material usage and construction costs (Distin et al., 2008).

The superior structural properties of high modulus material justify thickness reductions of 25 to 40% compared with conventional French materials (Sanders & Nunn, 2005, Nunn & Smith, 1997).

The main characteristic of these mixtures is high stiffness, obtained by using hard or very hard bitumens (low penetration grade binders in the range 10 to 35), which necessitates the use of finer grading (Nunn & Smith, 1997; Distin et al., 2008).

But since the use of such hard binders limits the workability of the mixtures, their use is justified in the case of very high traffic or for specific applications such as in the airport environment (Pasetto, 1998), (for the strengthening of taxiways and runways, Horak, 2008).

In essence, EME is hot-mix asphalt consisting of hard bitumen blended at high binder content with good quality, fully crushed aggregates in order to produce a (relatively) fine-graded mix with low air void content. EME is designed to combine good mechanical performance with durability and impermeability when well compacted. It is designed in the laboratory to yield high elastic stiffness, high permanent deformation resistance and high fatigue resistance, whilst also offering good moisture resistance and good workability, which are the four key parameters for long-life pavements (Distin et al., 2008). The designed material is considered to be very stable and consequently very heavy pneumatic-tyre rollers, weighing up to 45 tonnes, are regarded as essential for compaction (Nunn & Smith, 1997). The higher the modulus of elasticity, the higher the flexural rigidity and the dynamic resistance are. Also, the higher the modulus of elasticity the lower the thermal susceptibility and plastic deformation are. To achieve the required performance, i.e. high stiffness, fatigue resistance, high rutting resistance, resistance to brittle thermal cracking, ageing resistance and workability, appropriate asphalt mix designs were found which resulted in the first set of performance-based specifications NF P 98-140 published by AFNOR in 1992. Among the key components were the hard special bitumen grades, mostly 10/20 and 15/25 penetration, with characteristics that resulted in compromises between optimized thermal susceptibility and ageing resistance (Des Croix & Planque, 2004).

As mentioned above, EME is suited for applications such as a replacement layer or overlay, or also as a thinner structural layer when used in new construction. In particular:

- on heavily trafficked routes, particularly where traffic is slow and channelized, such as on major bus routes;
- in specific pavements subjected to heavy loads such as dedicated truck routes, loading bays and container terminals;
- in constrained (boxed-in) pavements, that is in areas where there are geometric constraints;
- on new pavements as a base course layer;
- in rehabilitation, where between 80 and 120 mm is milled off and replaced with EME, often surfaced with a very thin asphalt wearing course;
- on runways and taxiways (Distin et al., 2008).

In summary, from the economic point of view several advantages may arise from the use of EME, such as reduction of thickness from 3 to 7 cm, reduction of milled pavement and use of locally available stone aggregate.
Manufacturing and laying may be carried out at the same asphalt mixing plant and with the equipment used for traditional asphalt mixture (Paset-to, 1998).

1.4.1 In France

French mix design methodology consists of 5 basic steps (Distin et al., 2008).

- Step 1: Selection and identification of mix components (choice gradation and binder content). It is the crucial parameter for EME. The French Standard provides two classes of EME (EME 1 and EME 2), but EME 2 is the most widely used and it is obtained by using higher bitumen content (4.5-5% for EME 1 against 5.5-6% by weight of the aggregates for EME 2). The aggregate should be fully crushed to have high strength (Los Angeles value lower than 25-30%) and the maximum aggregate size can be 10, 14 or 20 mm (14 is the most widely used). The filler content is high (up to 10%). Figure 1.2 shows typical grading curves for EME, while Table 1.2 reports mix grading curve for EME 2 and diameter between 0 and 14 mm. The binder must be hard grade bitumen with a Penetration Index of around 0. The high level of bitumen plus filler and low air void content make it possible to have a good permanent deformation resistance (Brosseaud, 2012).

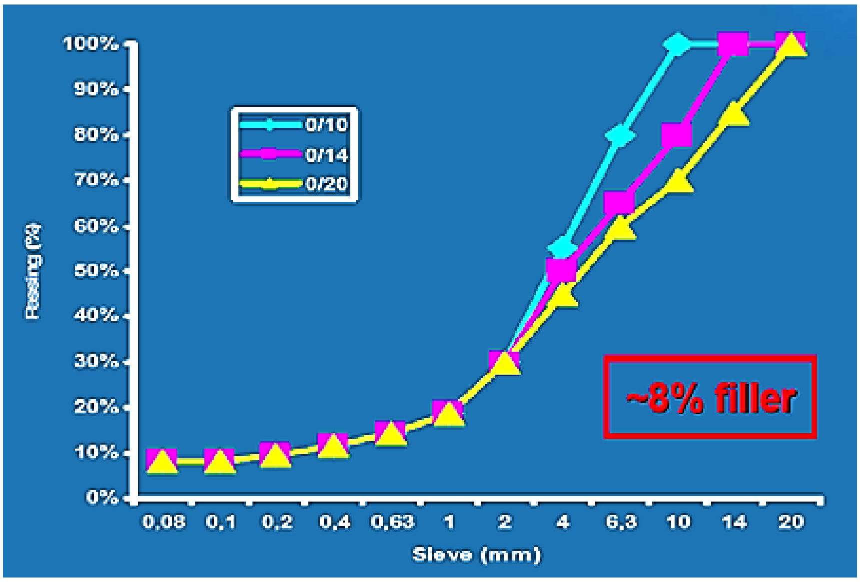

Fig. 1.2 Typical grading curves for EME (Distin et al., 2008)

Sieve (mm)	Passing (%)
14	94-100
10	72-84
6.3	50-66
4	40-54
2	28-40
0.08	7-10

Tab. 1.2 Mix grading curve for EME 2 0/14 (Pasetto, 1998)

- Step 2 (Level 1 Design): Assessment of workability (compactibility) and assessment of susceptibility to moisture damage (water sensitivity).
  The first is done by means of gyratory testing (ensuring that the mixing temperature is between 160 and 180°C and that the compaction temperature never drops below 145°C) and the second one by means of the Duriez test. If tests do not meet the requirements, it is necessary to go back to step 1 and change something about the components.
- Step 3 (Level 2 Design): Assessment of resistance to permanent deformation (rutting).
  The assessment of resistance to permanent deformation is carried out by means of a wheel-tracking test on slabs manufactured by

rolling-wheel compaction. The mix is subjected to 30000 unidirectional loads (frequency: 1 Hz, load: 5 KN, pressure: 0.6 MPa) at a test temperature of 60°C.

- Step 4 (Level 3 Design): Assessment of the elastic stiffness.
  The asphalt mix stiffness is determined by either a complex modulus test (sinusoidal loading on a trapezoidal or parallelepiped specimen) or a uniaxial tensile test (on a cylindrical or parallelepiped specimen). With preset frequency, time and loading values the test is started, and in it short-term deformations are induced in the sample. The modulus (stress-strain ratio) is computed for each basic test. Using time-temperature transposition, an elastic stiffness master curve is then developed.
- Step 5 (Level 4 Design): Determination of fatigue life.
  The tests are conducted at a temperature of 10°C using a loading frequency of 25 Hz.

The last word regarding the suitability of using high modulus asphalt is thus given by the complex modulus and fatigue line, obtained by dynamic tests such as compression (or bending) tests where a sinusoidal load is applied at different temperatures and frequencies. In conclusion the required characteristics are summarized in Table 1.3.

Characteristics	EME 1	EME 2
Granularity and average thickness	0/10 from 6 to 10 cm 0/14 from 7 to 12 cm 0/20 from 10 to 15 cm	0/10 from 6 to 10 cm 0/14 from 7 to 12 cm 0/20 from 10 to 15 cm
Minimum richness factor for asphalt content (k)	≥2.5	≥3.4
Binder content for 0/14 grading (%)	≥3.9	≥5.4
Water sensitivity test (Duriez test) r/R	≥0.70	≥0.75
Wheel-tracking test (60°C, 30000 cycles)	≤7.5%	≤7.5%
Complex modulus MPa (15°C, 10 Hz)	≥14000	≥14000
Fatigue test $\varepsilon_{10}{}^{6}$ microstrain (10°C, 25 Hz)	≥100	≥130
Void content (%)	≤10	≤6

Tab. 1.3 French Specifications for EME (Horak, 2008)

EME not only offers thinner layers, but also offers sustainable life long pavements (Horak, 2008).

1.4.2 In the other countries

According to a research project developed in 2007 by the CRR (Centre de Recherches Routieres in Belgium), which takes as its primary reference the French standard NF P 98-140, these mixtures were considered as a possible solution to contrast rutting in Belgium, due to increased heavy traffic. The primary objective of this study was to assess the technological feasibility, in terms of mix design, production and application of these mixtures very well-known abroad, especially in France, the country of origin, and still little used in Belgium. In other words, the aim was to provide the necessary knowledge in the study of the mix design of EME and the correct requirements to be included in the specifications by verifying that it was possible to produce this mixture with the materials commonly used in Belgium (De Backer et al., 2007).

There is doubtless similar interest in application of these mixtures in Italy (Moramarco, 2012; Pasetto, 1998), where there is not only heavy traffic, but also and above all a Mediterranean climate, which is more penalizing than the French and Belgian continental climate. Indeed, the use of EME would be particularly advantageous in warm climates (Espersson, 2004).

Table 1.4 gives an example of Italian grading curve for high modulus asphalt.

Sieve (mm)	Passing (%)
12.7	100
9.52	76-100
6.35	60-77
4.76	52-66
2	35-46
0.42	15-22
0.177	9-14
0.075	6-9

Tab. 1.4 Italian grading curve for high modulus asphalt (Pasetto, 1998)

Although for several years these mixtures have been used in many applications such as highways, urban roads and airport runways (Parracini et al., 2013), Italian standards does not yet cover the high modulus asphalt (Pasetto, 1998).

These mixtures are also of particular interest in other countries such as Australia (Petho & Denneman, 2013a; Petho & Denneman, 2013b; Petho et al., 2014; Guyot, 2013, Department of Transport and main roads, 2015, Le Bouteiller, 2016), United Kingdom (Nunn & Smith, 1994; Nunn & Smith, 1997; Sanders & Nunn, 2005), South Africa (Sabita, 2010; Nkgapele et al., 2012; Distin et al., 2008;), Spain (Pasetto, 1998), Poland, Morocco and Mauritius (Le Bouteiller, 2016).

Chapter 2

Case 1

2.1 General

Case 1 regards asphalt surface mixtures with improved performance using waste polymers via dry and wet processes and the tests were performed at the Road Materials Laboratory of the University of Guimarães (Portugal).

The results obtained indicated that polymer-modified mixtures showed similar or improved performance when compared to that of a conventional control mixture produced with harder virgin grade bitumen, not always available, or available at higher costs, in several countries. Thus, modifying asphalt mixtures with these plastic wastes can be an economic and ecological alternative for paving works. Moreover, the mixtures produced via the dry process showed increased water sensitivity and stiffness modulus properties. This holds out new possibilities for use of polymer-modified mixtures, especially in developing countries, since it widens the possibility of using locally available bitumens, of variable quality, for producing mixtures with higher performance. This can be achieved at real scale with no major extra costs because the dry process does not require modification of typical asphalt plants.

2.2 Materials

The mixture used in this study, an " AC 14 SURF ", is a conventional mixture commonly used in surface courses in which the maximum aggregate size (D_{max}) is 14 mm. The materials used to produce polymer-modified asphalt mixtures with HDPE (High Density Polyethylene) and EVA (Ethylene-Vinyl Acetate), as well as the conventional control mixture, namely the aggregates, the bitumen and the additives, are presented below. The methods used for mixture production and material testing are presented later.

2.2.1 Aggregates

The mineral aggregates used in the mixtures came from crushed granitic rocks (supplied by the Bezerras quarry at Guimarães, Portugal), while the filler was limestone (supplied by Omya, S.A. at Soure, Portugal).

The mixtures were obtained from the combination of different aggregate fraction sizes (6/14, 4/6 and 0/4) with the addition of mineral filler. For each fraction, the physical and mechanical characteristics are given in Table 2.1. Sieving operations were carried out in the laboratory in order to obtain the 10/14 fraction from the 6/14 fraction and the 2/4 and 0.5/2 fractions from the 0/4 fraction, respectively. In order to highlight the effect of polymer modification as well as that of the addition process, the mix grading curve was kept constant for the different mixtures studied; it is given in Table 2.2, together with the lower and upper limits typically required in the technical specifications for road works in Portugal (EP 2014).

Characteristic	Fraction 0/4	Fraction 4/6	Fraction 6/14	Filler	Unit	Standard
Density	2.66	2.65	2.66	2.7	Mg/m^3	EN 1097-6
Contents of fines	f_{16}	f_4	$f_{1.5}$			EN 933-1
Flattening Index		FI_{20}	FI_{15}			EN 933-3
Sand Equivalent (SE)	≥50				%	EN 933-8
Methylene Blue (MB)	≤5				%	EN 933-9
Los Angeles abrasion	LA_{30}	LA_{30}	LA_{30}			EN 1097-2

Tab. 2.1 Physical and mechanical characteristics of the aggregates used in this work

Sieve	Limit Lower	Limit Upper	filler	0.5/2	2/4	0/4	4/6	6/14	10/14	Total
#20	100	100	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
#14	90	100	100.0	100.0	100.0	100.0	100.0	91.0	86.6	96.0
#10	67	77	100.0	100.0	100.0	100.0	100.0	43.0	0.0	73.0
#4	40	52	100.0	100.0	67.9	90.8	11.7	6.0	0.0	46.7
#2	25	40	100.0	99.3	1.0	68.9	3.8	4.0	0.0	31.9
#0.5	11	19	100.0	0.0	0.1	36.5	2.6	3.0	0.0	16.8
#0.125	6	10	100.0	0.0	0.0	15.3	1.8	2.0	0.0	8.8
#0.063	5	8	96.0	0.0	0.0	9.8	1.1	1.0	0.0	6.3
Material Used			2.5%	3.0%	8.0%	36.0%	11.0%	29.0%	10.5%	100.0%

Tab. 2.2 Aggregate grading curve and envelope limits for AC 14 Surf mixtures

2.2.2 Bitumens

The base bitumen used in this study for polymer-modified mixtures was a 70/100 pen grade bitumen provided by Cepsa. The conventional mixture, assumed as the control mixture, was produced with a 35/50 pen grade bitumen, from the same supplier as the 70/100 bitumen. The characteristics of the two bitumens are detailed in Table 2.3.

The production of modified binders with the WET process was performed via modification of 70/100 pen grade bitumen by adding 5% of HDPE or EVA, by mass of binder, i.e. by replacing 5% of the total mass of binder with HDPE or EVA polymer. Later, the same proportions were used for the mixtures produced via the DRY process, in order to have the same amounts of bitumen and polymer in both methods. The polymer content was selected in the typical range, which usually produces a significant effect on the mechanical and rheological properties of the binder (Giuliani et al., 2009). Binder modification was performed in the laboratory, with a high-shear mixer at 7400 rpm at a constant temperature (T = 160°C). The characteristics of the resulting polymer-modified binders are given in Table 2.4.

Characteristics		Unit	Bitumen		Standard
			70/100	35/50	
Penetration at 25°C, pen		dmm	75.30	36.70	EN 1426
Ring and Ball Softening Point, $T_{R\&B}$		°C	48.70	54.80	EN 1427
Resilience		%	0.00	10.00	EN 13880-3
Viscosity	100°C	Pa·s	3.72	4.27	EN 13302
	110°C		1.82	2.33	
	120°C		0.99	1.29	
	130°C		0.57	0.76	
	140°C		0.36	0.47	
	150°C		0.23	0.30	
	160°C		0.16	0.20	
	170°C		0.11	0.14	
	180°C		0.08	0.10	

Tab. 2.3 Characteristics of the conventional bitumens

Characteristics		Unit	Bitumen		Standard
			70/100+HDPE	70/100+EVA	
Penetration at 25°C, pen		dmm	31.80	37.10	EN 1426
Ring and Ball Softening Point, $T_{R\&B}$		°C	67.90	61.10	EN 1427
Resilience		%	11.00	26.00	EN 13880-3
Viscosity	100°C	Pa·s	52.11	14.24	EN 13302
	110°C		17.54	7.16	
	120°C		5.34	3.71	
	130°C		2.04	2.10	
	140°C		1.23	1.22	
	150°C		0.85	0.76	
	160°C		0.65	0.50	
	170°C		0.45	0.34	
	180°C		0.32	0.24	

Tab. 2.4 Characteristics of the polymer-modified binders

A dynamic mechanical analysis of the several binders was conducted with a dynamic shear rheometer (DSR), making it possible to obtain the rheological properties in terms of complex modulus $|G^*|$ and phase angle δ for a reference temperature of 60 ºC, as shown in Figure 2.1. 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 LVE response of the material. The testing temperature ranged from 30°C to 80°C, while the testing frequency ranged from 0.01 to 100 Hz.

The isotherms obtained were used for determination of the Master Curves. These curves were obtained by applying the time-temperature superposition principle. In this connection, the shift in the frequency sweep tested at multiple temperatures by applying a multiplier (shift factor) to the frequency (or time) at which the measurement is taken made it possible to combine the individual isotherms of stiffness in order to form a single smooth curve of frequency or time versus stiffness (the master curve). The shift factor α_T was calculated and optimized according to the Arrhenius equation (Rowe & Sharrock, 2010; Celauro et al., 2010). 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 60°C for each binder.

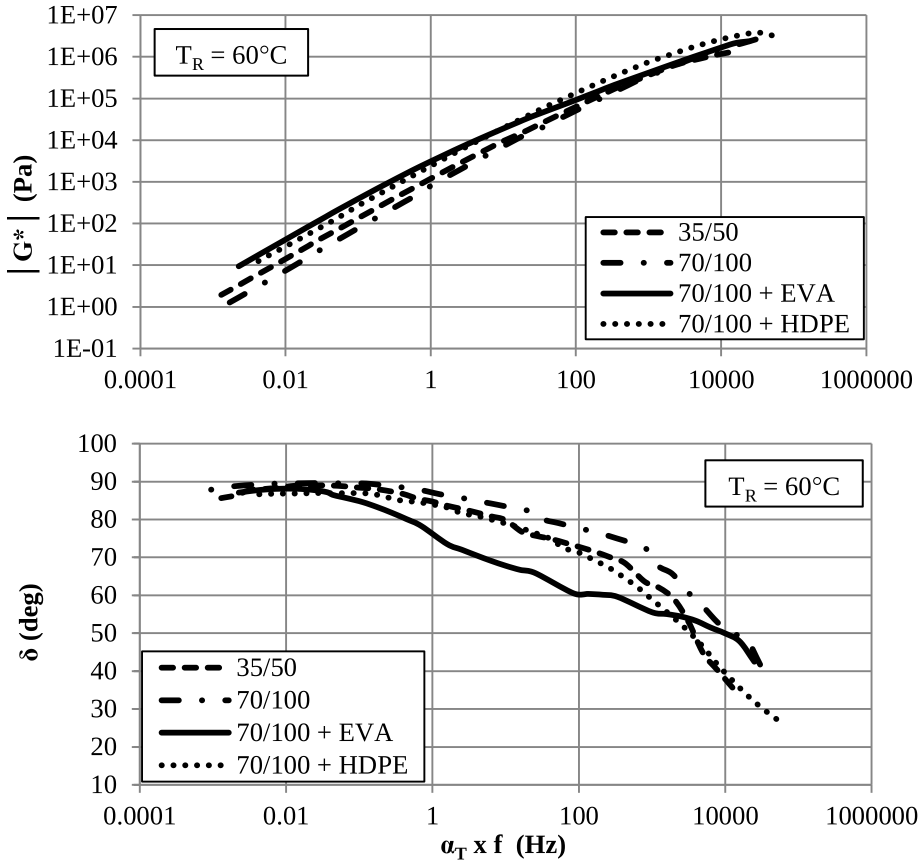

Fig. 2.1 Complex modulus and phase angle of the binders used

The modification of the 70/100 pen grade bitumen with 5% EVA or HDPE increased the stiffness moduli and reduced the phase angles in a wide range of frequencies. Based on the results given in Fig. 2.1, the polymer-modified binders became stiffer and more elastic compared not only to the unmodified 70/100 pen grade bitumen, but also to the 35/50 pen grade bitumen. This result confirms the advantages of polymer modification, as previously mentioned, which will later be assessed when studying the effect of polymer introduction in asphalt mixtures.

2.2.3 Additives

The plastic wastes used as additives, HDPE and EVA, provided by Gintegral, are both shown in Fig. 2.2. The former has a density of 0.81 g/cm^3 and dimensions between 0.125 and 4 mm, while the latter has a density of 0.93 g/cm^3 and granules with the same maximum dimension (4 mm). These density values were calculated according to ASTM D792, method A.

Differential scanning calorimetry (DSC) tests were carried out in accordance with the ISO 11357-3 standard on these polymer additives, in order to characterize their thermal behaviour. This test provided values of fusion temperatures corresponding to fusion enthalpies (ΔH) that are correlated with the degree of modification and compatibility between the plastics and the bitumen (Naskar et al., 2010). The test protocol was two heating ramps from -60°C to 180°C, and a cooling ramp from 180°C to -60°C between them, all of these at a heating/cooling rate of 10°C/min.

The DSC test results are shown in Figs. 2.3 and 2.4. It can be seen that the melting point of HDPE is T_{M_HDPE} = 130°C and the fusion enthalpy is ΔH_{HDPE} = 72 J/g. As expected, for EVA these values were much lower, T_{M_EVA}= 70°C and ΔH_{EVA} = 23 J/g. The fusion of the crystalline parts of the polymers occurred below the expected binder and mixture production temperature (165°C). As for the fusion enthalpy, these values indicate that EVA should be more compatible with the bitumen, but the degree of modification was higher when waste HDPE was added.

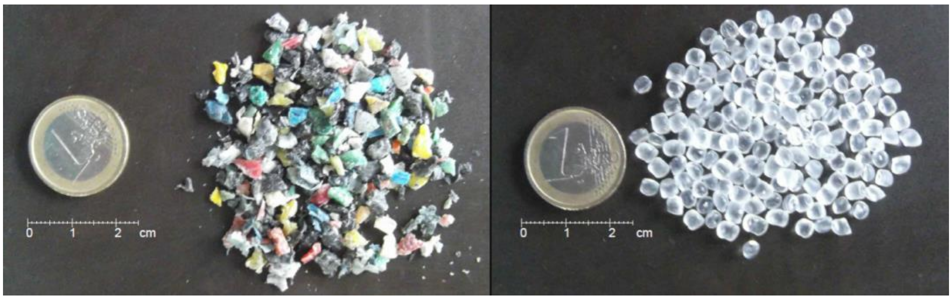

Fig. 2.2 Samples of HDPE (on the left) and EVA (on the right) used in this study

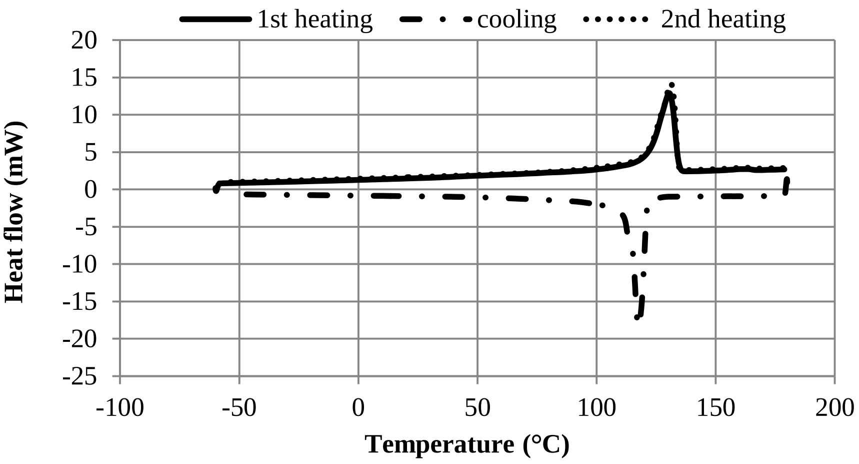

Fig. 2.3 DSC test results on HDPE

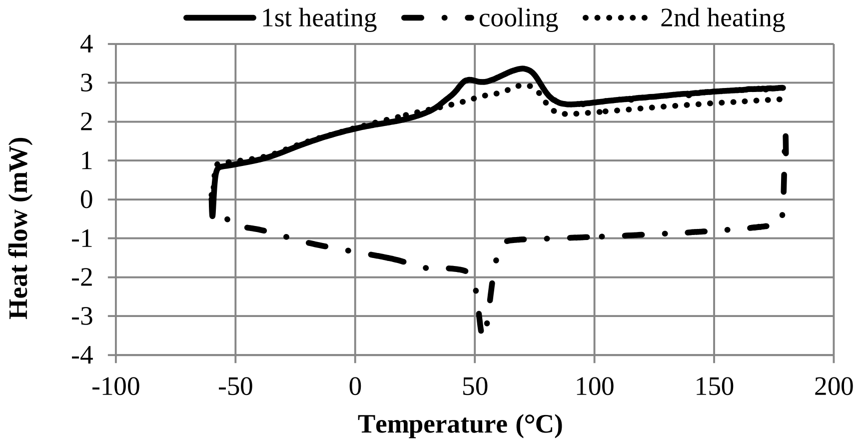

Fig. 2.4 DSC test results on EVA

2.3 Methods, results and discussion

Volumetric (mix design) and mechanical (water sensitivity, permanent deformation, stiffness modulus and fatigue cracking resistance) characterization of mix asphalt is essential and necessary in order to evaluate the performance of a road pavement. Thus, as mentioned above, the mechanical performance of a typical surface course mixture, modified with two different plastic wastes, both via the wet and dry processes, was evaluated in this study and compared to that of a conventional control mixture. Specifically, the five mixtures were named as follows:

- HDPEw (70/100+HDPE);
- HDPEd (70/100+HDPE);
- EVAw (70/100+EVA);
- EVAd (70/100+EVA);
- CONV (35/50).

In order to reduce any possible risk of instability of the polymer-bitumen blends, the asphalt mixtures produced with the WET process were manufactured immediately after binder production, with the traditional hot-mixing process. For the mixture produced with the DRY process, the polymer additive was directly added to the aggregate during the mixing process (see Figure 2.5). In accordance with the EN 12697-35 standard, the target mixing temperature for the control mixture with 35/50 pen grade bitumen was set equal to T = 165°C. For mixtures with modified binders, it was necessary to select a different temperature that would result in a similar viscosity. Thus, based on the results given in Tables 2.3 and 2.4 a temperature higher than 180°C would be necessary, but a mixing temperature of 180°C was selected in order to avoid any premature ageing of the base binder (70/110 pen). All these issues were taken into account in order to assure similar production/compaction conditions for the different mixtures.

Fig. 2.5 Addition of the additive: in the mixing (DRY process, on the left), into the bitumen (WET process, on the right)

The formulation test was the Marshall test with 3 different bitumen contents and CE equal to 75 (Compaction Effort is the number of blows per face during the production of specimens).
The following tests were carried out on optimized mixtures:

- water sensitivity test;

- rutting resistance test with wheel tracker machine (Wheel Tracking Test - WTT);
- dynamic test for complex stiffness modulus determination (Four Point Bending configuration - 4PB);
- dynamic test for fatigue resistance determination (Four Point Bending configuration- 4PB).

2.3.1 Mix design

A formulation test was carried out only on the mixtures produced with the WET process in order to choose a suitable bitumen content for the grading curve adopted (mix design).

The mix design was performed by the Marshall Method, according to the EN 12697-34 standard and using three bitumen contents (b'_1 = 4.5%, b'_2 = 5%, b'_3 = 5.5%, by mass of aggregates). Three specimens were used for each bitumen content. The results are reported in Tables 2.5-2.10 and Figures 2.6-2.12.

Specimen	Bitumen content b' (%)	Marshall Stability (KN)	Marshall Stability correct S_M (KN)	Deformation D (mm)	Shear Deformation D_t (mm)	Marshall Ratio R (KN/mm)
451 EVA		13.58	14.41	4.10	2.76	3.52
452 EVA	4.50	12.09	12.80	3.90	2.89	3.28
453 EVA		13.65	14.44	3.82	2.54	3.78
Average	4.50	13.11	13.89	3.94	2.73	3.53
501 EVA		14.41	15.74	3.93	2.74	4.00
502 EVA	5.00	14.55	15.89	4.57	3.03	3.48
503 EVA		13.99	14.81	4.05	2.95	3.65
Average	5.00	14.32	15.48	4.18	2.91	3.71
551 EVA		12.18	13.23	4.58	3.11	2.89
552 EVA	5.50	12.93	13.65	4.25	2.74	3.21
553 EVA		12.44	13.72	4.86	3.28	2.82
Average	5.50	12.52	13.53	4.56	3.04	2.97

Tab. 2.5 Marshall test results for the mixture with EVA (mechanical characteristics)

Specimen	Bitumen content b' (%)	Apparent specific gravity γ (g/cm³)	Maximum specific gravity BMT (g/cm³)	Porosity n (%)	Vb (%)	Vge (%)	VMA (%)	VFB (%)
451 EVA		2.40		4.3	9.7	86.0	14.0	69.1
452 EVA	4.50	2.39		4.8	9.6	85.6	14.4	66.9
453 EVA		2.39		4.5	9.7	85.9	14.1	68.5
Average	4.50	2.39	2.51	4.5	9.7	85.8	14.2	68.1
501 EVA		2.41		3.1	10.8	86.1	13.9	77.9
502 EVA	5.00	2.41		2.9	10.8	86.3	13.7	78.8
503 EVA		2.41		2.9	10.8	86.3	13.7	78.7
Average	5.00	2.41	2.49	3.0	10.8	86.2	13.8	78.5
551 EVA		2.42		2.0	11.9	86.2	13.8	85.9
552 EVA	5.50	2.42		1.9	11.9	86.2	13.8	86.3
553 EVA		2.42		2.0	11.9	86.1	13.9	85.6
Average	5.50	2.42	2.47	1.9	11.9	86.2	13.8	85.9

Tab. 2.6 Marshall test results for the mixture with EVA (physical characteristics)

Specimen	Bitumen content b' (%)	Marshall Stability (KN)	Marshall Stability correct S_M (KN)	Deformation D (mm)	Shear De-formation D_t (mm)	Marshall Ratio R (KN/mm)
451 HDPE		13.22	13.36	4.34	3.10	3.07
452 HDPE	4.50	14.80	15.50	3.89	2.45	3.98
453 HDPE		13.30	14.15	3.25	1.85	4.36
Average	4.50	13.77	14.34	3.83	2.47	3.81
501 HDPE		14.27	15.26	4.36	2.79	3.50
502 HDPE	5.00	14.17	14.62	3.99	2.54	3.66
503 HDPE		14.40	14.93	4.07	2.78	3.67
Average	5.00	14.28	14.93	4.14	2.70	3.61
551 HDPE		15.36	16.30	4.41	2.91	3.69
552 HDPE	5.50	14.78	15.24	4.37	2.66	3.48
553 HDPE		14.71	16.06	4.22	2.70	3.80
Average	5.50	14.95	15.87	4.34	2.76	3.66

Tab. 2.7 Marshall test results for the mixture with HDPE (mechanical characteristics)

Specimen	Bitumen content b' (%)	Apparent specific gravity γ (g/cm^3)	Maximum specific gravity BMT (g/cm^3)	Porosity n (%)	Vb (%)	Vge (%)	VMA (%)	VFB (%)
451 HDPE		2.39		4.3	9.7	86.0	14.0	69.5
452 HDPE	4.50	2.38		4.3	9.7	86.0	14.0	69.1
453 HDPE		2.38		4.3	9.7	86.0	14.0	69.2
Average	4.50	2.38	2.49	4.3	9.7	86.0	14.0	69.3
501 HDPE		2.39		3.9	10.7	85.4	14.6	73.3
502 HDPE	5.00	2.39		4.1	10.7	85.2	14.8	72.2
503 HDPE		2.40		3.7	10.7	85.6	14.4	74.1
Average	5.00	2.39	2.49	3.9	10.7	85.4	14.6	73.2
551 HDPE		2.40		2.1	11.8	86.0	14.0	84.7
552 HDPE	5.50	2.40		2.2	11.8	86.0	14.0	84.6
553 HDPE		2.40		2.1	11.9	86.1	13.9	85.0
Average	5.50	2.40	2.45	2.1	11.9	86.0	14.0	84.8

Tab. 2.8 Marshall test results for the mixture with HDPE (physical characteristics)

Specimen	Bitumen content b' (%)	Marshall Stability (KN)	Marshall Stability correct S_M (KN)	Deformation D (mm)	Shear De-formation D_t (mm)	Marshall Ratio R (KN/mm)
451 CONV		16.70	17.86	3.53	2.43	5.07
452 CONV	4.50	16.25	16.93	4.10	2.88	4.13
453 CONV		16.48	16.87	3.97	2.83	4.25
Average	4.50	16.48	17.22	3.86	2.71	4.48
501 CONV		16.07	16.62	3.70	2.35	4.49
502 CONV	5.00	16.76	16.98	4.10	2.78	4.14
503 CONV		16.66	17.27	3.94	2.74	4.39
Average	5.00	16.50	16.96	3.91	2.62	4.34
551 CONV		13.98	15.02	4.05	2.48	3.71
552 CONV	5.50	14.99	15.78	4.26	3.12	3.71
553 CONV		16.83	17.91	4.21	2.68	4.26
Average	5.50	15.26	16.24	4.17	2.76	3.89

Tab. 2.9 Marshall test results for the conventional mixture (mechanical characteristics)

Specimen	Bitumen content b' (%)	Apparent specific gravity γ (g/cm³)	Maximum specific gravity BMT (g/cm³)	Porosity n (%)	Vb (%)	Vge (%)	VMA (%)	VFB (%)
451 CONV		2.39		4.8	9.6	85.6	14.4	66.9
452 CONV	4.50	2.40		4.3	9.7	86.0	14.0	69.3
453 CONV		2.39		4.7	9.7	85.7	14.3	67.4
Average	4.50	2.40	2.51	4.6	9.7	85.8	14.2	67.9
501 CONV		2.40		4.2	10.7	85.1	14.9	71.6
502 CONV	5.00	2.40		4.3	10.6	85.0	15.0	71.0
503 CONV		2.41		4.0	10.7	85.3	14.7	72.8
Average	5.00	2.40	2.51	4.2	10.7	85.1	14.9	71.8
551 CONV		2.41		2.7	11.8	85.5	14.5	81.1
552 CONV	5.50	2.41		2.7	11.8	85.5	14.5	81.3
553 CONV		2.42		2.4	11.8	85.8	14.2	83.2
Average	5.50	2.41	2.48	2.6	11.8	85.6	14.4	81.9

Tab. 2.10 Marshall test results for the conventional mixture (physical characteristics)

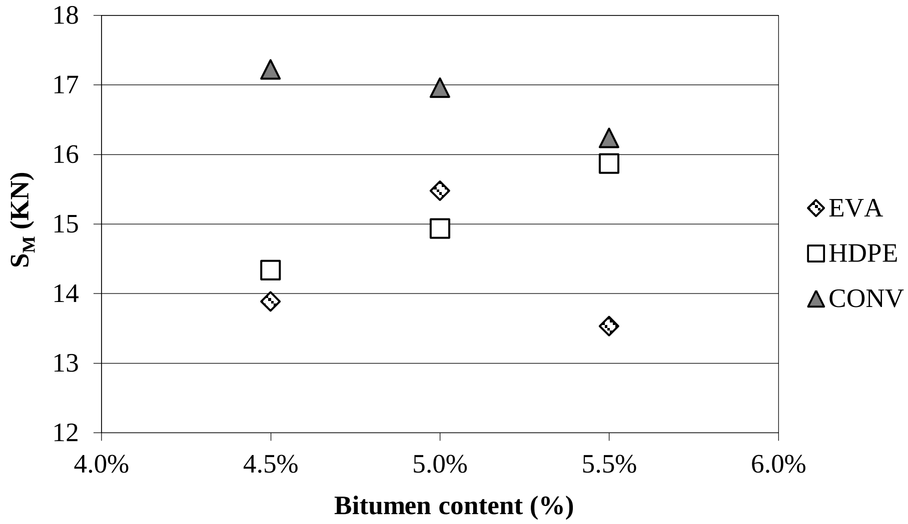

Fig. 2.6 Marshall Stability for bitumen content

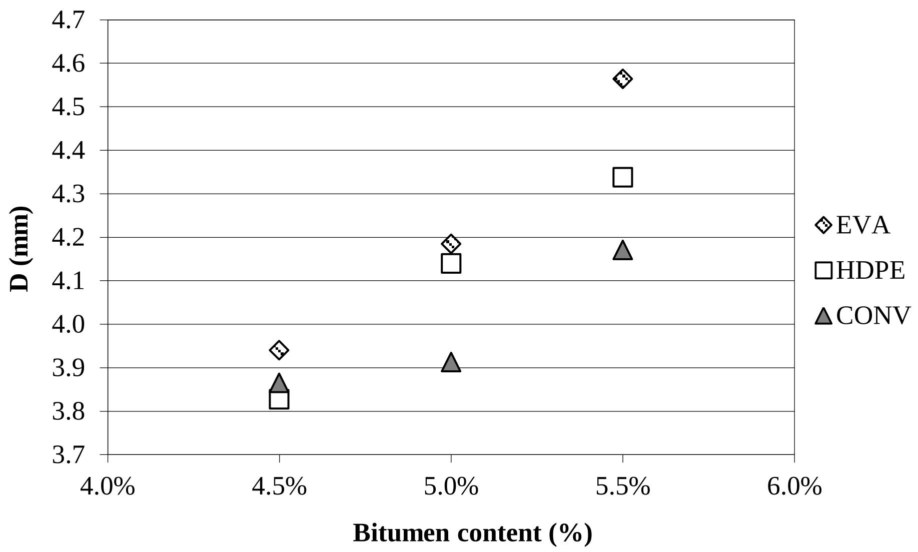

Fig. 2.7 Deformation for bitumen content

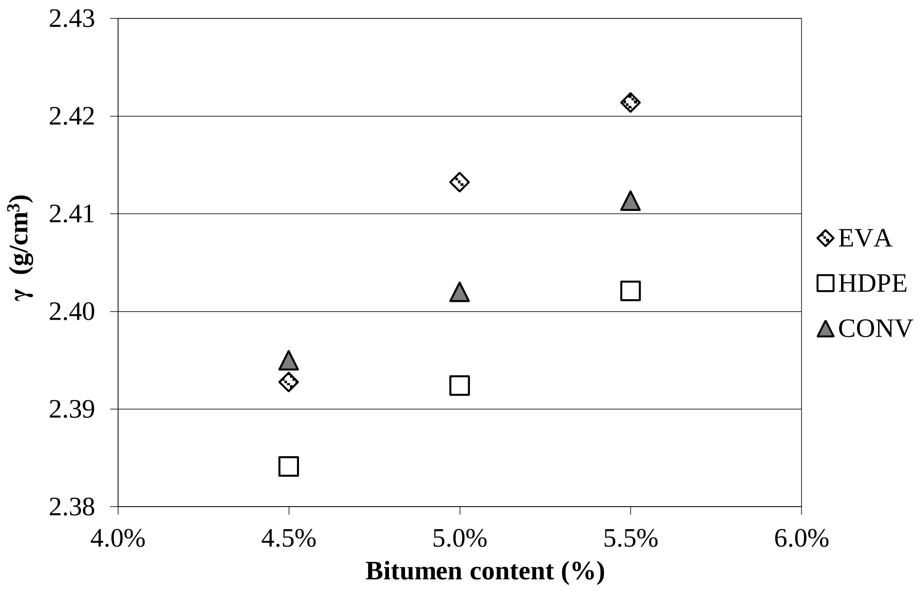

Fig. 2.8 Apparent specific gravity for bitumen content

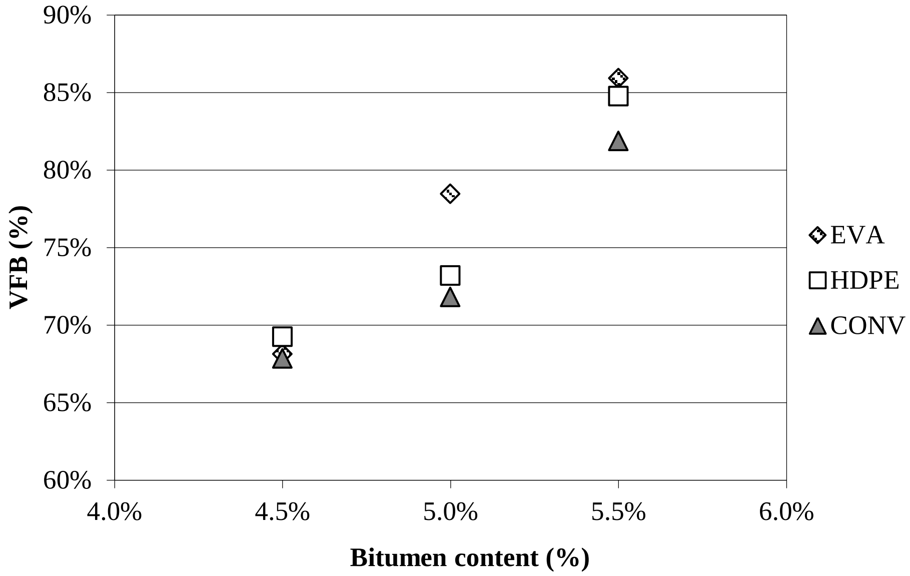

Fig. 2.9 Voids Filled with Bitumen (VFB) for bitumen content

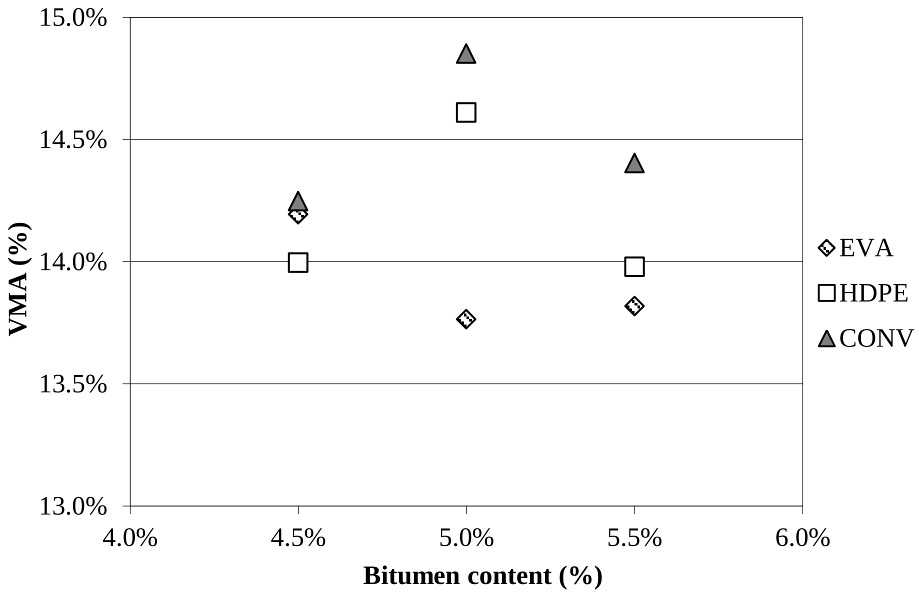

Fig. 2.10 Voids in mineral aggregate (VMA) for bitumen content

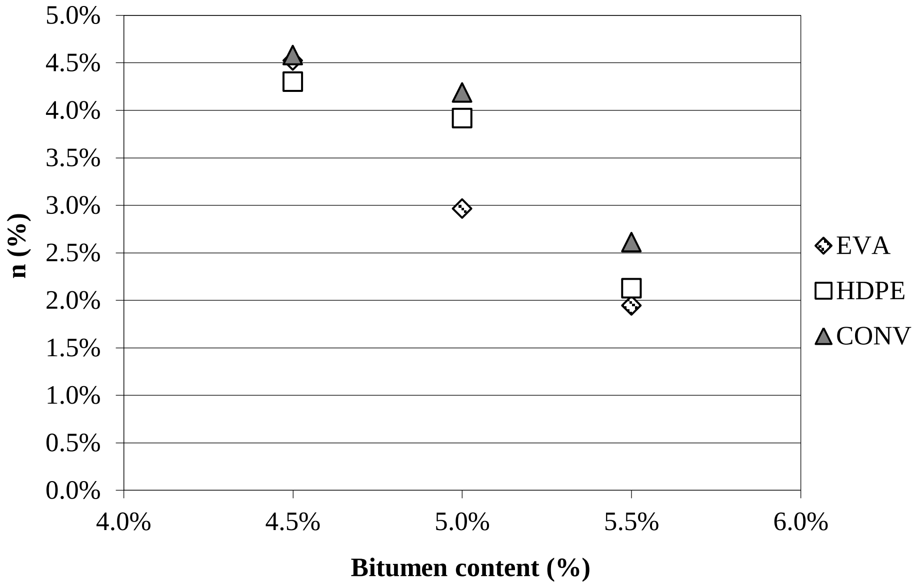

Fig. 2.11 Porosity for bitumen content

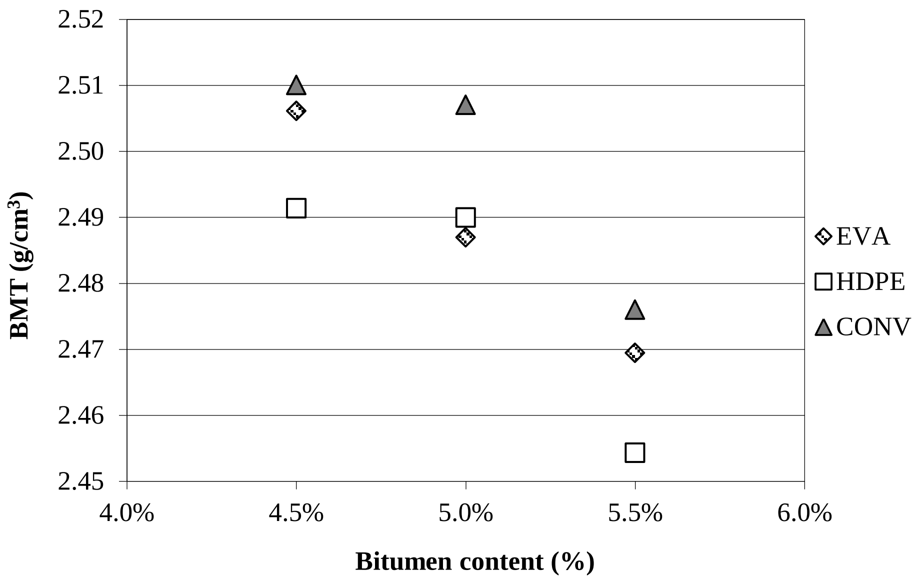

Fig. 2.12 Maximum specific gravity for bitumen content

For these mixtures the requirements are as follows (EP, 2014):

- Marshall Stability S_M = 7.5÷15 KN;
- deformation D = 2÷4 mm;
- Marshall Ratio R > 3 KN/mm;
- voids in mineral aggregate VMA > 14%;
- porosity n = 3÷5%.

Based on the results, a bitumen content of 5% was suitable for production of optimized mixtures, that is 5% binder content (4.75% bitumen and 0.25% polymer) and 95% aggregate content, by mass of total mixture, were obtained for the optimized mixture produced with the WET process. In order to avoid adding more variables into the study, the same binder content was used for the mixture produced with the DRY process.

2.3.2 Water sensitivity

The mixtures studied were tested for water sensitivity according to the EN 12697-12 standard. This test comprised assessment of the indirect tensile strength (ITS), carried out according to EN 12697-23, of two identical groups of three specimens conditioned in different environments (dry and wet with the application of vacuum). In that period, one group was kept dry and the other was immersed in water, in order to determine the influence of water on the weakening of the bond between aggregates and binder and, consequently, on the strength of the mixture.

The test procedure also included evaluation of the indirect tensile strength ratio (ITSR) between the average results for both groups of specimens, in order to assess the water sensitivity of the mixture, as reported in Equation (2.1).

$$\mathrm{ITSR} = \left(\frac{\mathrm{ITS_w}}{\mathrm{ITS_d}}\right) \times 100 \tag{2.1}$$

where “ $\mathrm{ITS_w}$ ” is the average ITS of the wet group of specimens while “ $\mathrm{ITS_d}$ ” is the average ITS of the dry group of specimens. The higher the ITSR, the lower the loss of mechanical resistance due to water action, which is typically selected as the acceptance criterion for control of the construction (Praticò, 2007).

This property was evaluated by subjecting specimens produced with a Marshall impact compactor to indirect tensile tests. The average results obtained from each group of three wet and dry specimens are reported in Figure 2.13.

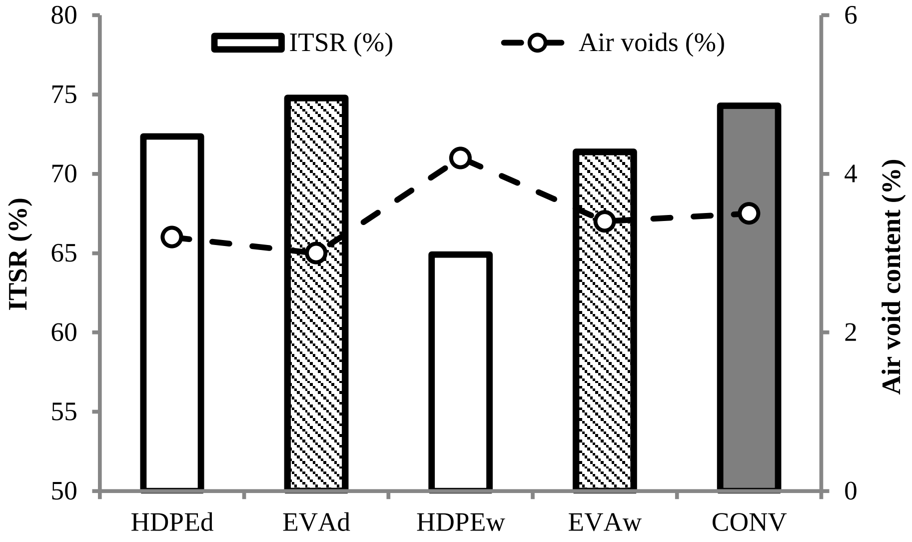

Fig. 2.13 Water sensitivity tests results (ITSR; air void content)

Regarding the effect of water on mechanical performance, the ITSR values were generally close to 75%, which means that these mixtures can be considered to have low water sensitivity. These good results were obtained for all mixtures and thus the mixtures with polymers offered performance that is comparable to that of the control mixture, produced with a harder bitumen. Further, given the repeatability of the test results, their dispersion is such that the performance can be considered almost homogeneous.

It is also possible to observe that all mixtures presented similar volumetric properties in terms of air void content. The exception was the HDPEw mixture, which presented a slightly higher air void content as a result of the higher viscosity of the modified binder used in that mixture. Moreover, regarding the effect of the production process, the mixtures produced with the DRY process offered better water sensitivity results compared to the mixtures produced with the WET process (the mixture produced with the WET process containing HDPE presented the lowest ITSR value, 65%, probably due to the abovementioned higher viscosity of this polymer-modified binder in association with the higher fusion enthalpy of HDPE). With regards to the effect of the polymer additives, in general mixtures with EVA proved to have better performance compared to that of mixtures with HDPE.

In order to detail the reliability of the estimation procedure for these results, Figure 2.14 also shows the error bars, based on the mean μ and standard deviation σ of indirect tensile strength values of dry specimens, associated with a 95% confidence level.

First, it was noticed that, for all the mixtures studied, the ITSd values were much higher than the typical minimum range values (700-1350 KPa) required for surface courses (MIT, 2002). It was also observed that in all cases, the polymer-modified mixtures presented lower ITS values than the conventional mixture. This reduction, on average 33%, may be explained by the use of a much softer base bitumen in the polymer-modified binders. The higher dispersion phase in the colloidal system of this softer bitumen may not be sufficiently balanced by the introduction of polymers in the binder molecular structure, at least regarding its behaviour at the temperature used in this test (15°C), even though the performance at higher temperatures is clearly satisfactory.

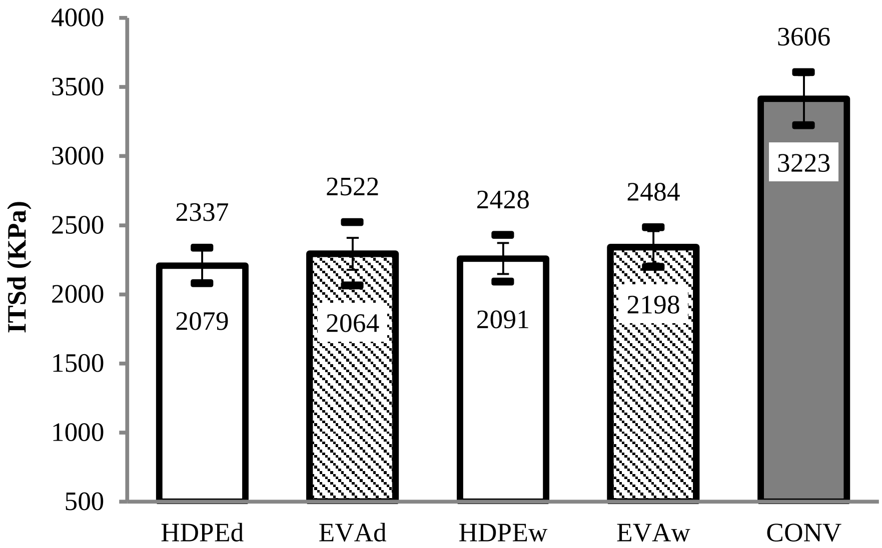

Fig. 2.14 Average indirect tensile strength, ITSd, of dry specimens

2.3.3 Permanent deformation resistance

One of the most common forms of distress of asphalt concrete pavements is rutting. It is defined as progressive accumulation of permanent deformation of each layer of the pavement structure under repetitive loading (Tayfur et al., 2007).

In order to assess the rutting resistance of the mixtures studied, wheel tracking tests were carried out according to EN 12697-22 method B.

The wheel tracking test is used for determining the susceptibility of hot mix asphalt to deformation under a load by measuring the rut depth formed by repeated passes of a loaded wheel at a fixed temperature.

Two slabs were produced for each type of mixture, with dimensions of 300 × 300 × 40 mm. The values for the two tests were averaged. The tests were performed at a temperature T = 50°C, as being representative of the hottest summer days in Portugal, up to 10000 cycles or a maximum deformation of 20 mm, at a frequency f = 0.44 Hz and a load p = 700 N.

The main results obtained in this test were the wheel tracking slope (WTS air), which is the deformation rate per thousand cycles (calculated between the 5000th and 10000th loading cycle), and the total rut depth at the end of the test. Mixtures with lower values of these parameters are known to have higher resistance to deformation at high temperatures.

Regarding the rut resistance results, all the polymer-modified mixtures (using both the dry and the wet process) proved to perform better than the conventional mixture, as can be noticed in Figure 2.15. Indeed, the mean rut depth values (values presented in the right hand side of the graph) were reduced by more than half (for EVA-modified mixtures) and by up to a third (for HDPE-modified mixtures) in comparison with the conventional asphalt mixture, i.e. the rut depth values were decreased by around 70% to 76%.

Regarding the effect of the production process, it is not as substantial as that observed when using different polymers. Furthermore, the mixtures produced with the WET process generally had slightly better performance than those produced with the DRY process, taking into account the lower mean wheel tracking slope (WTS) values of these mixtures observed in Fig. 2.15. When analysing the evolution of the rut depth throughout the test, it could also be seen that the EVA-modified mixtures produced with the WET or DRY process did not show substantial differences among them. This may be due to the lower fusion enthalpy of this plastic waste, which indicates higher compatibility with the binder, allowing an adequate interaction between the polymer and the binder even if this is carried out in shorter periods (as in the DRY mixing process).

Finally, regarding the polymers, the mixtures with HDPE were more resistant to permanent deformation than those with EVA, which is confirmed by both the reduced mean rut depth and the WTS values. This result is due to the higher stability of the HDPE-modified binder at high temperatures, as previously observed in the softening point test results (the HDPE-modified binder showed a $T_{R\&B}$ of 67.9 C against the 61.1 C value of the EVA-modified binder).

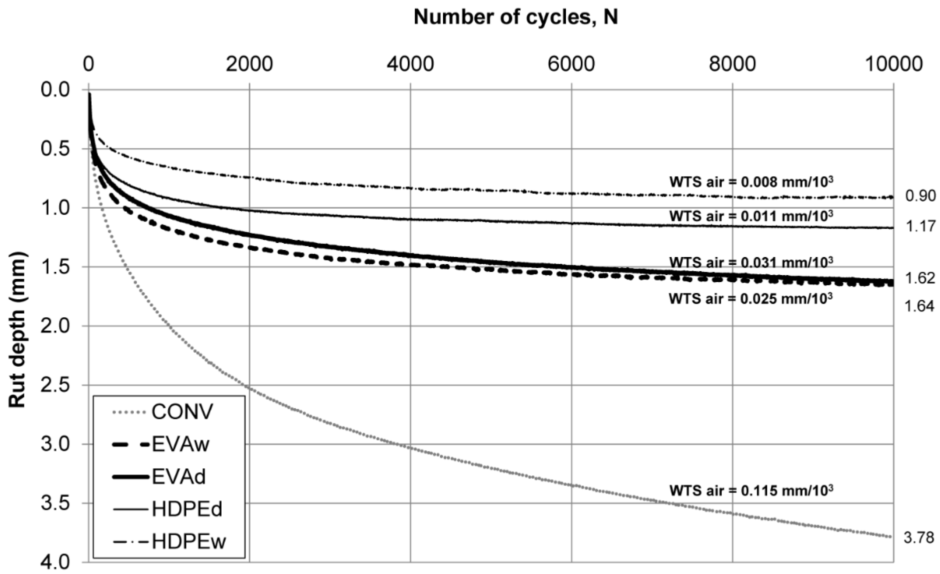

Fig. 2.15 Evolution of the permanent deformation of the studied mixtures in the WTT

2.3.4 Stiffness modulus

The stiffness modulus and fatigue cracking resistance of the mixtures studied were obtained by dynamic tests on prismatic beams taken from slabs produced by a roller compactor. Among the different possible testing protocols available to assess these properties (Dondi et al., 2013), the tests were carried out using a four-point bending (4PB) test device imposing sinusoidal loading with a constant strain amplitude (strain-controlled method, see Figure 2.16).

A prismatic beam rests on two smooth bilateral supports that allow the rotation and translation of the specimen in the direction of the axis. The load is applied at two points equidistant from the centre with the same intensity (see Figure 2.17). During the test, the beam is held in place by four clamps and a repeated haversine (sinusoidal) load is applied to the two inner clamps with the outer clamps providing a reaction load. This setup produces a constant bending moment over the center portion of the beam (between the two inside clamps). The deflection caused by the loading is measured at the center of the beam. The number of loading cycles to failure can then give an estimate of an HMA mixture's fatigue life (Maggiore et al., 2012).

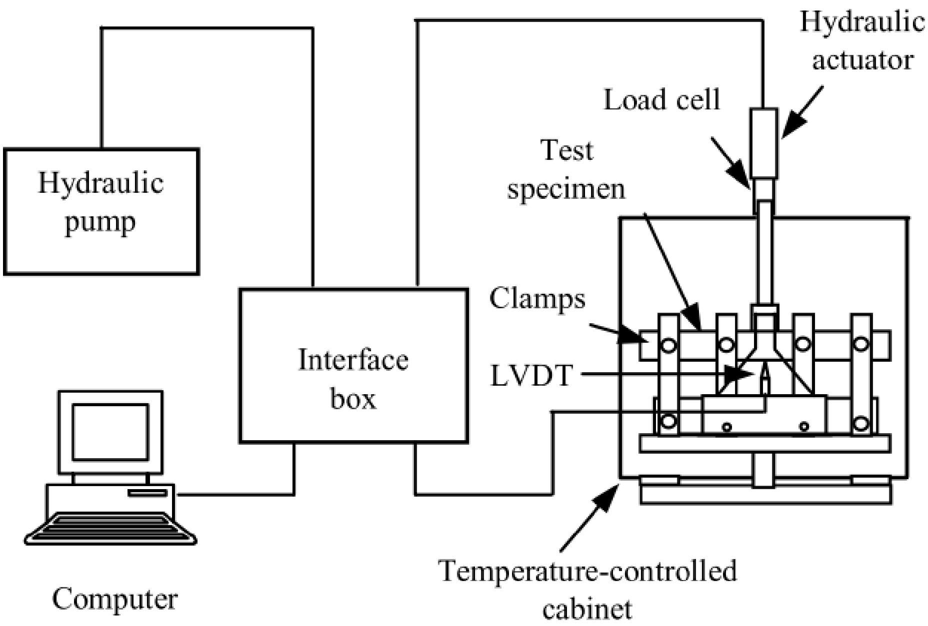

Fig. 2.16 Scheme of a 4 PB test device (Artamendi & Khalid, 2004)

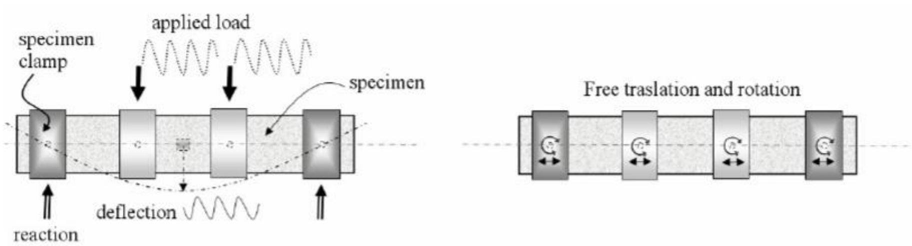

Fig. 2.17 Loading scheme of the specimens in a 4 PB test (Coni et al., 2008)

In general, regarding controlled-strain tests, failure of the specimens is difficult to define, since the stresses within the specimen decrease during the test, as the specimens gets progressively weaker due to the accumulation of damage, and total failure of the specimen is unlikely to occur (Coni et al., 2008).

Stiffness modulus tests were carried out on beams with dimensions 305 x 50 x 50 mm at four different temperatures (T = 0, 10, 20 and 30°C) and at a range of frequencies f = 0.1 to 10 Hz, according to EN 12697-26 annex B. The strain amplitude ε was set equal to 100×10^{-6}, within the linear response of the materials studied.

The structural performance of pavements is directly related to the mechanical behaviour of asphalt mixtures, which can be characterized by the stiffness modulus |E*| and phase angle δ.

|E*| is defined as the ratio of the amplitude of the sinusoidal stress σ and the amplitude of the sinusoidal strain ε, as follows (Rowe et al., 2009):

$$|E^*| = \frac{\sigma}{\varepsilon} = \sqrt{E'^2 + E''^2} = \sqrt{(|E^*| \cos \delta)^2 + (|E^*| \sin \delta)^2} \qquad (2.2)$$

where E' is the storage modulus, E'' is the loss modulus and δ is the lag between the load and deformation signal.

Figure 2.18 depicts the master curves of the stiffness modulus |E*| for the different mixtures at the reference temperature T_R = 20°C over a range of reduced frequencies between 10^{-3} and 10^{4} Hz obtained by applying the time-temperature superposition principle.

In general, the stiffness modulus increases with the test frequency (high frequencies are known to correspond to higher load application speeds which means lower loading times). The opposite occurs for the phase angles. These results are typical of the whole range of temperatures and strain levels (Picado-Santos et al., 2003).

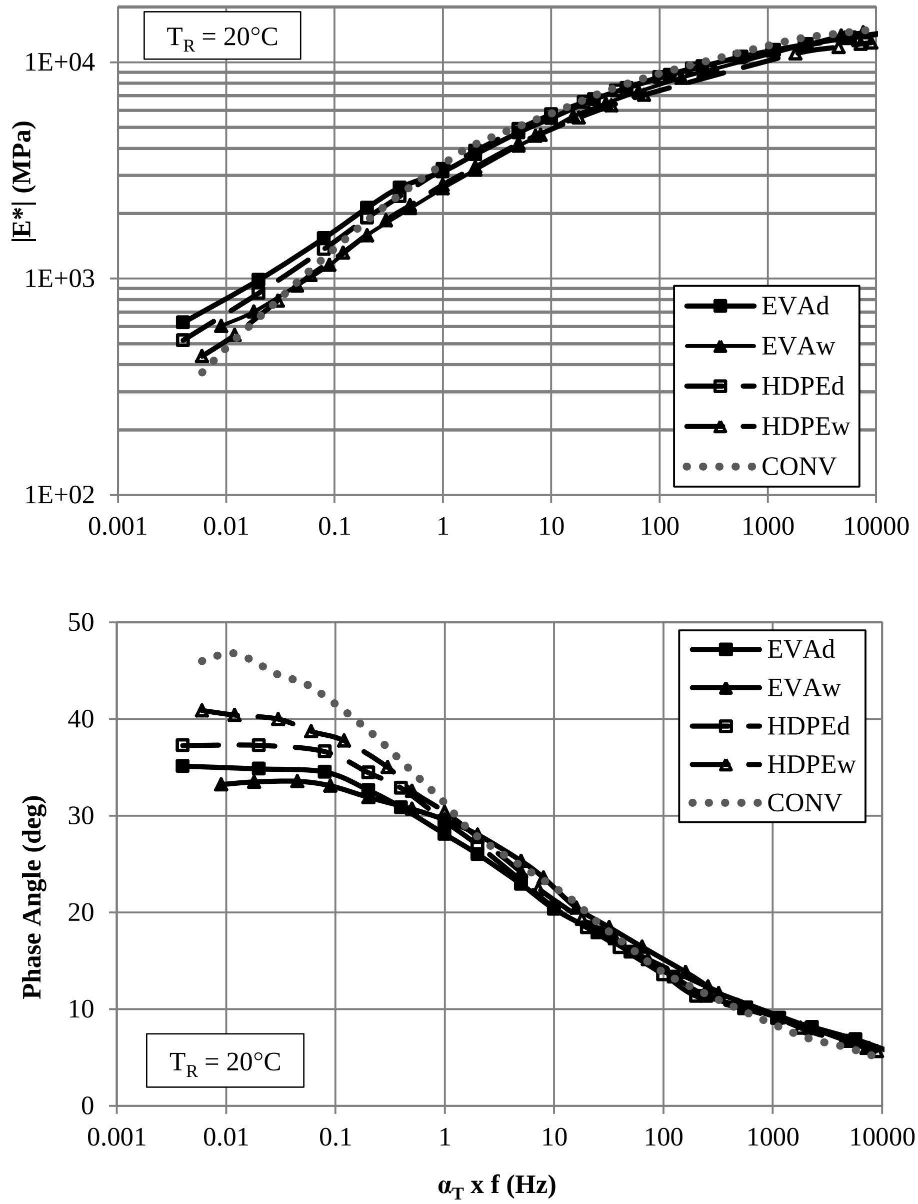

Fig. 2.18 Stiffness modulus and phase angle master curves of the mixtures studied

Regarding the effect of introducing different polymer/plastic wastes on the stiffness modulus of the asphalt mixtures, it can be seen that the values were similar for the different mixtures studied.

Comparing the results for the polymer-modified mixtures with those for the conventional mixture, it can be observed that they had higher stiffness modulus values at low frequencies (or high temperatures) and lower stiffness modulus values at high frequencies (or low temperatures).

A similar crossing trend can be observed from the master curves of the phase angle, even though higher stiffness values correspond to lower phase angle values. Thus, the influence of the addition of polymers can be clearly observed in both graphs presented in Fig. 2.18. At high temperatures (low frequencies) the stiffness is higher and the elastic component is larger in the modified mixtures due to the presence of a stabler molecular structure provided by the polymer. Moreover, at lower temperatures the lower stiffness values of the modified mixtures are provided by the behaviour of the softer base bitumen used, as previously explained, which can result in mixtures with higher flexibility.

In particular, the stiffness modulus values of the mixtures produced with the DRY process are similar to or slightly higher than those of the mixtures produced with the WET process. The success of the DRY process, already observed during the water sensitivity tests, can be associated with the use of a very soft base bitumen, which is more capable of creating effective interactions with both the aggregates and the polymers when this method is used. Regarding the influence of the polymers, both presented similar master curves, even though the master curves of the EVA-modified mixtures showed slightly higher stiffness values, as a consequence of a better interaction of this polymer with bitumen due to its lower fusion temperature.

2.3.5 Fatigue cracking

Fatigue is an important failure mode of pavement structures. Consequently, a correct description of this phenomenon is of great importance. Bituminous materials in roads are subjected to short-term loading each time a vehicle passes. If sufficiently high, the loading results in a loss of rigidity of the material and can lead to failure by accumulation in the long term. During fatigue cracking, two phases of the degradation process are usually considered: initiation and propagation phases. The first phase corresponds to degradation resulting from damage that is uniformly spread in the material. Hence, this phase is manifested by the initiation and propagation of a “ micro-crack ” network (in a diffuse way), which results in a decrease in the macroscopic rigidity (modulus). In the second phase, from the coalescence of micro-cracks, a " macro-crack " appears which propagates within the material. (Di Benedetto et al., 2004).

Fatigue tests were carried out at T = 20°C at a constant frequency f = 10 Hz, according to EN 12697-24, annex D. The fatigue cracking resistance of the different mixtures was determined by subjecting the specimens to loading procedures analogous to those of the stiffness modulus tests (strain controlled method), but using higher strain levels (not necessarily within the linear response of the materials studied). In fact, different strain levels were applied in order to obtain the fatigue life equation of the mixtures studied.

The fatigue test results for the mixtures studied made it possible to obtain the regression lines (Wöhler curves, given by Equation (2.3)), which relate the strain level applied in the test (ε) with the number of loading cycles (N). These are based on a classical approach, which assumes that damage to the specimen accumulates during testing until failure occurs. This is considered to happen when asphalt mixtures stiffness reduction is 50% of its initial value (Picado-Santos et al., 2009). These fatigue regression laws are presented in Figure 2.19.

$$\varepsilon = a \times N^{-b} \tag{2.3}$$

where:

- ε is the strain level applied during the test;
- a is a constant depending on the physical and mechanical characteristics of the material, test temperature and frequency;
- b is the slope of the fatigue lines;
- N is the number of loading cycles.

Based on these fatigue laws it is possible to obtain some fatigue indicators for the mixtures studied, as summarized in Table 2.11. The determination coefficient R^2 is important for assessing the statistical quality of the laws obtained. This indicator showed significantly high values for most of the mixtures studied, although the mixture modified with EVA using the DRY process showed a slightly lower R^2. This could be associated with a slightly increased difficulty in obtaining totally homogeneous distribution of the polymer within the mixture in the DRY process.

The parameter ε_6 is the admissible strain level at N = 10^6 loading cycles, being one of the main indicators used to characterize the fatigue resistance of asphalt mixtures in European standards.

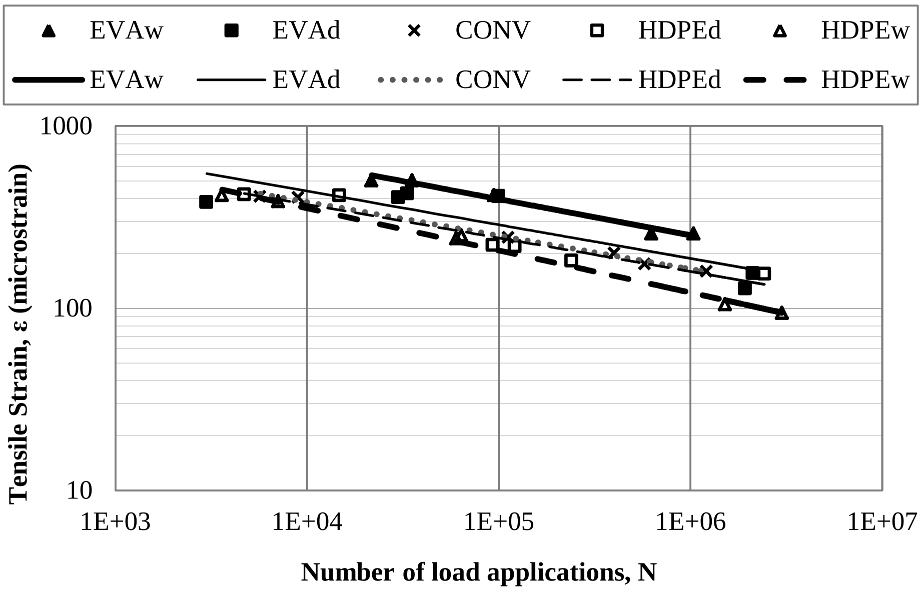

Fig. 2.19 Fatigue regression laws of the mixtures studied

Parameter	HDPEd	EVAd	HDPEw	EVAw	CONV
a	2027.6	2410.5	2978.6	3890.6	2093.5
b	0.184	0.185	0.231	0.198	0.184
R^2	0.91	0.75	0.99	0.97	0.99
ε_6	159.58	187.11	122.47	252.36	164.77

Tab. 2.11 Fatigue indicators obtained from the regression laws of the mixtures studied

By comparing the fatigue laws, it can be noticed that both the mixtures with EVA offered better fatigue performance than that of the conventional mixture, since they were shifted upwards, evidencing that better performance was obtained after introducing EVA. In particular, the EVA mixture produced with the WET process presented higher fatigue resistance than that produced with the DRY process. This improved fatigue performance is in agreement with previously presented results, confirming the importance of selecting polymers with better affinity with the bitumen and highlighting the advantage of using waste EVA polymers in asphalt mixture modification.

On the other hand, the mixtures with HDPE did not show the same improved fatigue cracking performance. Indeed, although the mixture produced with the DRY process offered similar performance to that of the conventional mixture, the HDPE mixture produced with the WET process was the mixture with the worst fatigue performance of all those evaluated experimentally in this study. It must be noted that when using the DRY process, the interaction between the base binder and HDPE can be lower than that obtained when using the WET process, resulting in a more flexible binder, which can explain the better fatigue performance.

In any case, all admissible strain values ε_6 (>130 μstrain) can be considered satisfactory for road paving applications, showing that both polymers and both incorporation processes are viable solutions.

2.4 Conclusions

This experimental study led to important conclusions from a comparison between two different processes (WET and DRY) used for production of modified asphalt mixtures with waste polymers.

When comparing the performance of polymer-modified asphalt mixtures with a conventional mixture, it was concluded that the ones with polymers are able to offer similar or better performance in terms of water sensitivity. Also regarding the stiffness modulus, the addition of polymers generally results in beneficial behaviour since it slightly increases stiffness at high temperatures and reduces stiffness at low temperatures.

This is also valid for rutting resistance (the rut depth values were reduced by more than half for EVA-modified mixtures and by up to a third for HDPE-modified mixtures); in terms of fatigue resistance, mixtures produced with HDPE had similar or slightly worse performance than the conventional mixture, while mixtures with EVA showed significant improvement.

When comparing the effect of the DRY vs. WET mixing processes on mixture performance, on the one hand it was concluded that all mixtures were volumetrically equivalent and showed similar performance regarding both stiffness modulus and water sensitivity, except the HDPEw mixture, which showed poorer water sensitivity (probably because this mixture presented a slightly higher air void content, as a consequence of the higher viscosity of the modified binder); on the other hand, the WET process seemed to have a slightly better effect on the mixtures' rutting resistance and on the fatigue cracking resistance of the EVA-modified mixture. The soft-base bitumen selected for production of all polymer-modified mixtures may have been a key factor to obtain very good performance in those solutions, since it allowed a better interaction with polymers and aggregates, in both the WET and in the DRY processes.

This study presents a series of innovative approaches and results when studying asphalt mixture modification. In particular, waste polymers were successfully used instead of virgin polymers; the DRY process of incorporation showed interesting performance, in comparison to the traditional WET process, mainly due to the use of a soft-base bitumen; the importance of selecting appropriate polymers was highlighted, since EVA generally presented better performance, due to its better interaction with bitumen; however, waste HDPE can be an interesting alternative when rutting resistance is the main performance concern or when its higher availability may dominate the decision criteria.

As a final remark, it can be mentioned that the WET process requires modification of the binder with specific equipment for blending bitumen and polymers at high temperatures, while no modification is needed for the DRY process. Therefore, the DRY process can become a good alternative to the usual binder polymer modification, since it can be easier and cheaper to implement, and results in mixtures with similar performance to those produced with the WET process. This could be particularly important for developing countries, since it widens the possibility of using locally available bitumens (of variable quality) and asphalt plants for producing mixtures with higher performance.

Chapter 3

Case 2

3.1 General

Case 2 is an experimental study concerning the development and optimization (mix design) of special asphalt mixtures for binder and base courses, also with additives specifically engineered.

The focus was on the optimization of the mixtures making use of ordinary aggregate and bitumen (as locally available) instead of the very hard bitumen typically prescribed, also aiming to improve the traditional mixture as made possible by suitable polymers.

For this purpose, the mixtures were subjected not only to conventional tests for mix design purposes, such as the Marshall test and the gyratory compactor test, but advanced tests for performance evaluation such as rutting resistance and fatigue resistance were investigated via small-scale testing with a wheel-tracking machine and four-point bending tests respectively. The dynamic modulus was also evaluated by triaxial cell tests at the University of Palermo’s Road Materials Laboratory.

The test results allowed important conclusions about the use of additives for these mixtures. In particular, mixtures with additives had better performance than ones without additives regarding permanent deformation resistance and more or less the same performance in terms of stiffness modulus. Moreover, the presence of additives allows the use of less hard bitumens (more easily available in Italy) than those typically used for these special formulations.

3.2 Materials

The special asphalt mixtures that are the object of this study are known in the technical literature as EME (enrobés à module élevé), that is, as already mentioned in the first chapter, high-modulus asphalt mixtures, with good performance both in terms of stability and durability.

The materials used to produce these mixtures will now be presented.

3.2.1 Aggregates

For the production of these mixtures it appeared appropriate to refer to aggregates with a diameter between 0 and 16 mm, a suitable range for binder and base courses. The aggregates used in the mixtures were crushed limestone from a quarry, whose composition and physical and mechanical properties are given respectively in Table 3.1 and Table 3.2.

Sieve (mm)	Passing (%)				
	Fractions				
	a_2 (20/25)	a_3 (10/15)	a_4 (6/10)	a_5 (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

Tab. 3.1 Composition of the aggregates' available fractions

Characteristics	Fractions				
	a_2	a_3	a_4	a_5	filler
Bulk specific weight, g/cm^3 (EN 1097-7)					2.85
Apparent specific weight, g/cm^3 (EN 1097-6)	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 (CNR 65)	0.80	0.79	0.71	0.73	
Absorption coefficient (EN 1097-6)	0.64	0.51			

Tab. 3.2 Physical and mechanical characteristics of the available aggregates

The typical dense graded high-modulus asphalt was obtained for the composition of the fractions provided by implantation and by appropriate sub-fractions (see Figure 3.1).

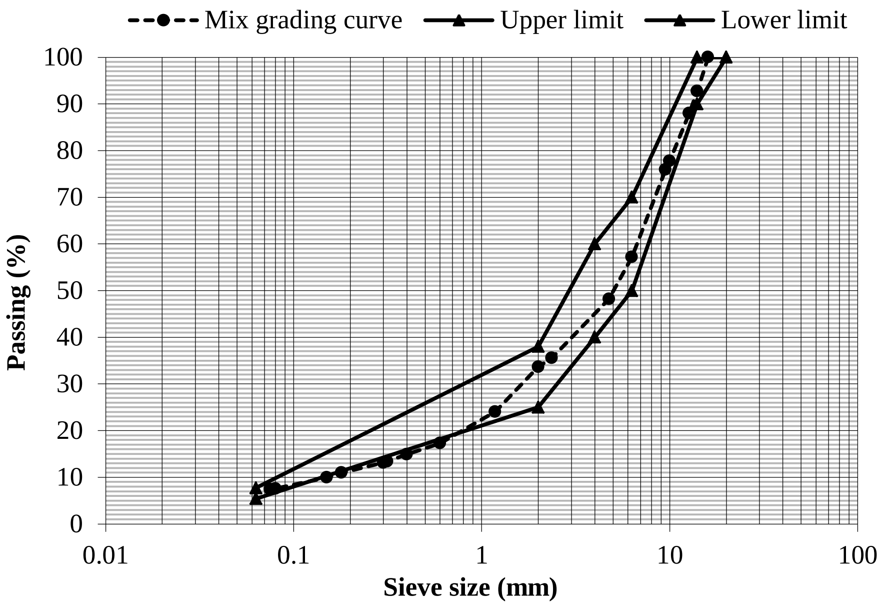

Fig. 3.1 Mix grading curve

The mixture of aggregates was then subjected to the pycnometer method for determination of the real specific weight and the value was 2.842 g/cm^3, according to the EN 1097-3:1999 standard.

3.2.2 Bitumens

The bitumens used in this research were neat bitumen 50/70 (more easily available in contexts such as in Italy) and near bitumen 35/50; their characteristics are reported in Tables 3.3-3.4.

A dynamic mechanical analysis of these binders was conducted with a dynamic shear rheometer (DSR), making it possible to obtain the rheological properties in terms of complex modulus $|G^*|$ and phase angle δ for a reference temperature of 30°C, as shown in Figure 3.2. 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 LVE 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.

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

Tab. 3.3 Characteristics of 50/70 pen grade bitumen

Characteristic	Unit	Value	Standard
Specific weight at 25°C	g/cm^3	1.033	EN 3838
Penetration at 25°C	dmm	35	EN 1426
Ring and Ball Softening Point	°C	56	EN 1427
Viscosity at 60 °C	Pa·s	224	EN 13302
Viscosity at 100 °C	Pa·s	9.88	EN 13302
Viscosity at 130 °C	Pa·s	1.11	EN 13302
Viscosity at 160 °C	Pa·s	0.24	EN 13302
Viscosity at 180 °C	Pa·s	0.11	EN 13302
Mixing temperature (η = 0.17 Pa·s)	°C	161	EN 13302
Compaction temperature (η = 0.28 Pa·s)	°C	150	EN 13302
After RTFOT:			
Change in mass	%	0.062	EN 12607-1
Penetration at 25°C	dmm	25	EN 1426
Ring and Ball Softening Point	°C	61	EN 1427
Viscosity at 60 °C	Pa·s	417	EN 13302

Tab. 3.4 Characteristics of 35/50 pen grade bitumen

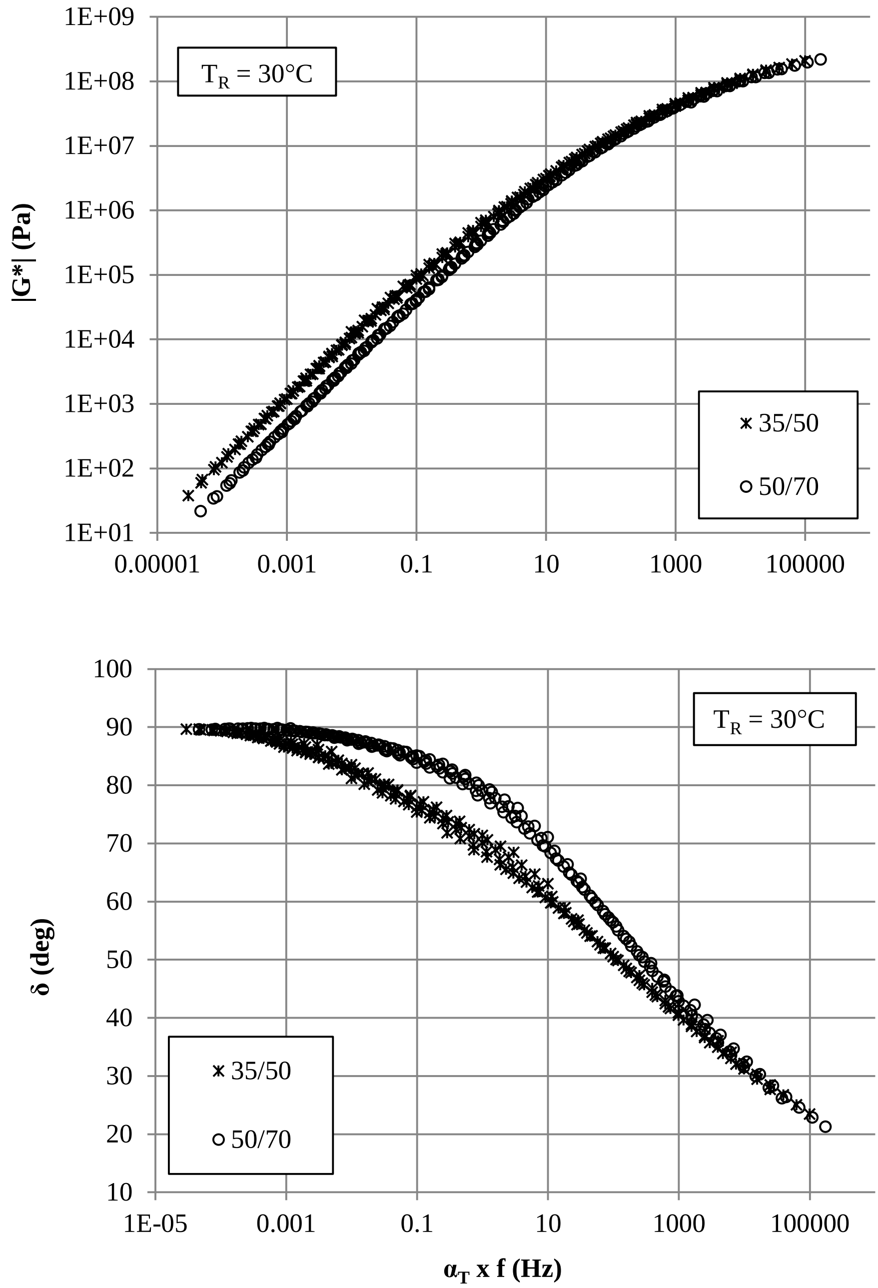

Fig. 3.2 Complex modulus and phase angle master curves of the binders used

Based on the results given in Fig. 3.2, 35/50 pen grade bitumen is stiffer and more elastic when compared to 50/70 pen grade bitumen. This result confirms the advantages of using low penetration grade binders, as previously mentioned, for high-modulus asphalt mixtures.

3.2.3 Additives

The additives used were a particular polymeric compound of selected polymers named SuperPlast (SP), a polyfunctional polymeric system named PPS, both provided by Iterchimica s.r.l., and waste plastics, as shown in Figure 3.3.

a) b) c)

Fig. 3.3 Additives used in this study: a) SP, b) waste plastics, c) PPS

SP is a compound of low-density polyethylene (LDPE) and ethylene-vinyl acetate (EVA) 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 recommended (Iterchimica S.r.l, 2015). The physical properties 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.

Waste plastics come from the recycling of tarps for greenhouses. The specific weight value is 0.94 g/cm^3.

PPS is a compound of cellulosic and glass fibers and plastomeric polymers, as well as SP. It was designed to give more resistance and durability to the road pavement. In fact it is able to act simultaneously on several properties, causing a physical and chemical modification in bituminous mixtures, acting in particular on the characteristics of the bitumen (penetration, softening point, viscosity) and creating a micro-structural reinforcement of the bitumen film. A dosage of 0.2-0.6% on weight of aggregates is recommended (Venturini et al., 2015).

The physical properties provided by the manufacturer are:

- colour: from pale grey to dark brown;
- average diameter: 4÷6 mm;
- fixed residue at 500°C: 20÷30%;
- residual moisture: ≤ 10%;
- apparent density: 0.45÷0.60 g/cm^3.

A differential scanning calorimetry (DSC) test was carried out on SP and waste plastics in accordance with the ISO 11357-3 standard, which makes it possible to characterize thermal behaviour because it provides the thermal transitions of a polymer, that is temperature and enthalpy values corresponding to glass transition (Tg), melting point (Tm) and crystallization (Tc) through heating from -60°C to 180°C, cooling from 180°C to -60°C and again heating (see Figures 3.4-3.5).

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 and thus a DSC curve is like the one in Figure 3.5 a (endo up), but in some instruments it is possible to find a plot like the one shown in Figure 3.5 b (exo up). The test results are shown in Figures 3.6-3.7.

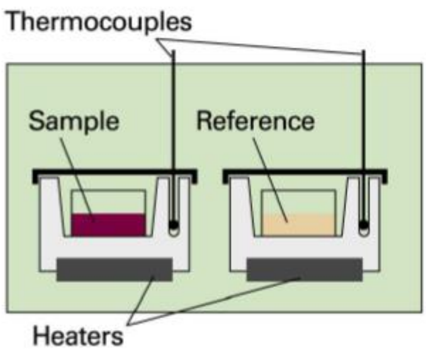

Fig. 3.4 A differential scanning calorimeter

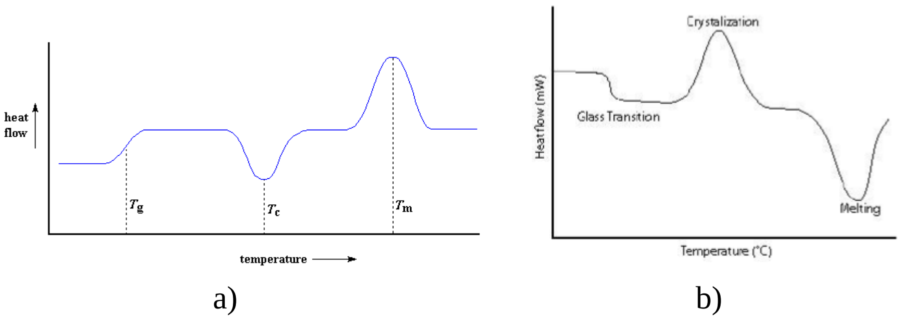

Fig. 3.5 Features of a DSC curve: a) endo up, b) exo up

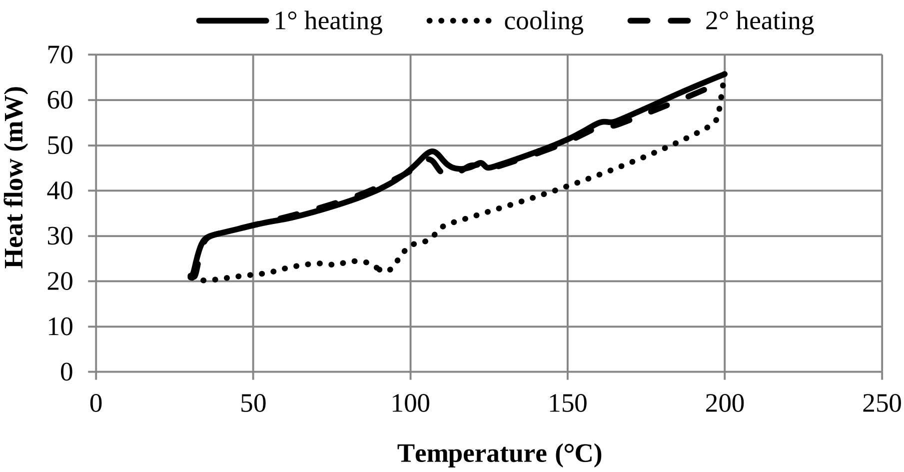

Fig. 3.6 DSC test results on SP

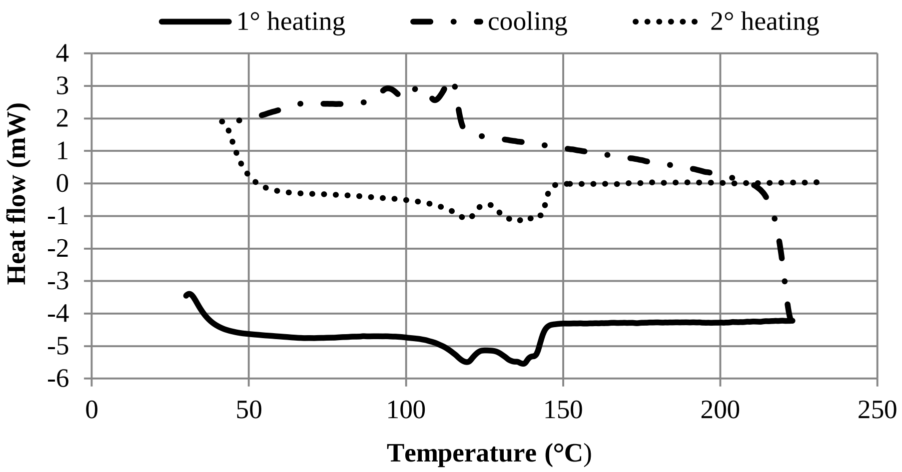

Fig. 3.7 DSC test results on waste plastics

As can be seen in Fig. 3.6, there are three peaks: the first is 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 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 because they refer to crystallization of polymers at low temperatures (Ashraf, 2014, METTLER TOLEDO, 2013).

This test confirms that SP is a low-density polyethylene with a small quantity of high-density polyethylene and polypropylene, while based on Fig. 3.7 can be said that waste plastics are a mix of high density polyethylene and polypropylene.

These additives have low affinity with the bitumen and therefore, for practical needs, it is much more advantageous to add additives 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 (Celauro et al., 2001a; Celauro et al., 2004).

3.3 Methods, results and discussion

The experimental study was developed on the following mixtures:

- EME SP.0 (50/70);

- EME SP (50/70+SP);
- EME P (50/70+waste plastics);
- EME PPS (50/70+PPS);
- EME CONV(35/50).

The mix design was achieved by carrying out the following:

1. Marshall test with 4 percentages of bitumen and CE equal to 75 blows per face, according to the EN 12697-34 standard;
2. compactibility test with a gyratory compactor (D = 150 mm), in accordance with the EN 12697-31 standard.

Finally, tests on optimized mixtures were performed in order to assess the following properties:

- rutting resistance, according to the EN 12697-22 standard, method B;
- complex stiffness modulus, according to the EN 12697-26 standard, annexes B and D;
- fatigue resistance, according to the EN 12697-24 standard, annex D.

3.3.1 Mix design

The first step in the formulation of these mixtures was applying the Marshall Method, according to the EN 12697-34 standard, only to the traditional mixture (without additives) produced with 50/70 pen grade bitumen, in order to assess the physical and mechanical characteristics typically considered in the Italian Specifications.

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 four specimens for each percentage were produced, for a valid repetition. 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 (γ_t) 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 Marshall test results are reported in Table 3.5 and Figure 3.8, where S is the Marshall Stability, F is the Marshall Flow, which is the corresponding displacement, and R the Marshall Ratio S/F.

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

Tab. 3.5 Marshall test results

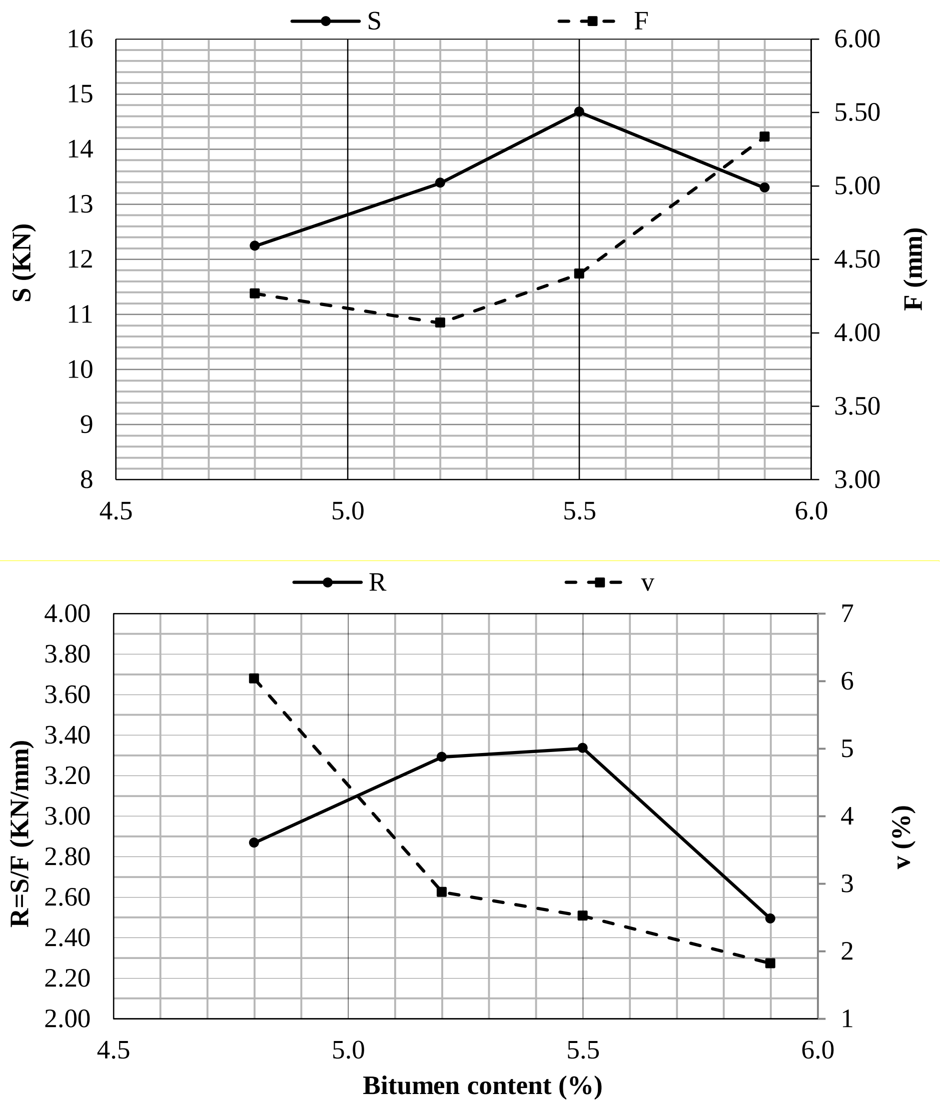

Fig. 3.8 Marshall values at different bitumen contents

As can be seen from the graphs above, regarding the Italian specifications, such as the example in Table 3.6 (MIT, 2002), the requirements are respected for bitumen contents between 4.8 and 5.1%.

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

Tab. 3.6 Marshall test results in accordance with MIT Specification

After some information obtained from the Marshall method the second step in the mix design of these mixtures was to consider some compactibility characteristics also considered by the Superpave method introduced in 1992 by the Strategic Highway Research Program SHRP (Cominsky et al., 1994).

The mixtures were therefore subjected to gyratory compaction, according to the EN 12697-31 standard, using a superpave gyratory compactor.

This compactor is designed to compact prepared HMA specimens at a constant consolidation pressure, a constant angle of gyrations 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 by Equation (3.1).

$$\sigma = \left(\frac{S}{P}\right) \tag{3.1}$$

where S is the shear stress given by Equation 3.2 and P is the ram pressure given by Equation (3.3).

$$S = \left(\frac{F \times d}{V}\right) \tag{3.2}$$

where F is a vertical force applied in order to achieve the gyration angle α during compaction, d is the lever arm distance and V is the specimen volume.

$$P = \left(\frac{R}{A}\right) \tag{3.3}$$

where 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 (Bayomy & Abdo, 2007).

The shear diagram is shown in Figure 3.9.

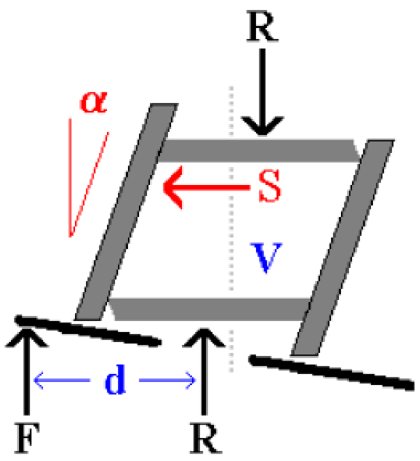

Fig. 3.9 Shear diagram (Bayomy & Abdo, 2007)

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.

In general, an increasing trend of the shear ratio σ in the initial stage of compaction (approximately, in the first 50 rpm) 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 (Roberts et al., 1996).

This test gives a good idea of the job-site density values, according to course thickness. Conducted ahead of the other mechanical tests, this test is used to make a preliminary selection or screening of mixes, and for optimizing the asphalt mix composition.

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 SuperPlast contents (SP.0 = 0% of polymer, SP.3 = 0.3% of polymer, SP.6 = 0.6% of polymer, by weight of mineral aggregates).

Later, the specimens with SP being subject to bleeding due to the excessive binder content, it was also decided to carry out the test, for the percentage 0.3 of SP, on two specimens with two lower bitumen contents (4.3, 4.6).

After performing the tests on these two mixtures, based on the results the test was also carried out on the conventional control mixture (with near bitumen 35/50) and the mixture with PPS, using the same bitumen contents as mentioned above (4.9, 5.1 and 5.4 %) and the polymer contents selected respectively for the traditional mixture without additives and the mixture with SP (0% of polymer and 0.3% of polymer, by weight of mineral aggregates).

Instead regarding the mixture with waste plastics different bitumen contents (4.6, 4.9 and 5.1 %) and different plastic contents (0.3, 0.5, 0.7, 1) were investigated.

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 (see Table 3.7).

From the values reported in Tab. 3.7 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.

Shear ratio values, determined during the test as mentioned above and automatically recorded by the equipment, are reported in Figures 3.10-3.11.

Fig. 3.10 presents the shear ratio lines of the mixtures studied, except for the mixture with waste plastics reported in Fig. 3.11 in order to make it as readable as possible.

Mixture	b	p	% $G_{mm} = C_1 + k * \log(N)$		
	%	%	C_1	K	R^2
EME 4.3/SP.3	4.3	0.3	0.8009	0.0757	0.9980
EME 4.6/SP.3	4.6	0.3	0.8072	0.0739	0.9984
EME 4.9/SP.0	4.9	0.0	0.7972	0.0753	0.9989
EME 4.9/SP.3	4.9	0.3	0.8141	0.0788	0.9952
EME 4.9/SP.6	4.9	0.6	0.8278	0.0748	0.9900
EME 5.1/SP.0	5.1	0.0	0.8190	0.0710	0.9988
EME 5.1/SP.3	5.1	0.3	0.8168	0.0786	0.9940
EME 5.1/SP.6	5.1	0.6	0.8274	0.0755	0.9880
EME 5.4/SP.0	5.4	0.0	0.8141	0.0788	0.9952
EME 5.4/SP.3	5.4	0.3	0.8345	0.0741	0.9837
EME 5.4/SP.6	5.4	0.6	0.8366	0.0742	0.9702
EME 4.6/P.3	4.6	0.3	0.8232	0.0743	0.9896
EME 4.6/P.5	4.6	0.5	0.8154	0.0751	0.9913
EME 4.6/P.7	4.6	0.7	0.8108	0.0769	0.9929
EME 4.6/P1	4.6	1.0	0.8463	0.0660	0.9627
EME 4.9/P.5	4.9	0.5	0.8253	0.0744	0.9897
EME 4.9/P.7	4.9	0.7	0.8257	0.0731	0.9865
EME 4.9/P1	4.9	1.0	0.8540	0.0638	0.9518
EME 5.1/P1	5.1	1.0	0.8589	0.0617	0.9439
EME 4.9/CONV	4.9	0.0	0.7989	0.0789	0.9969
EME 5.1/CONV	5.1	0.0	0.8278	0.0763	0.9889
EME 5.4/CONV	5.4	0.0	0.8224	0.0805	0.9756
EME 4.9/PPS.3	4.9	0.3	0.8125	0.0797	0.9920
EME 5.1/PPS.3	5.1	0.3	0.8145	0.0798	0.9939
EME 5.4/PPS.3	5.4	0.3	0.8251	0.0787	0.9859

Tab. 3.7 Values of workability and self-densification

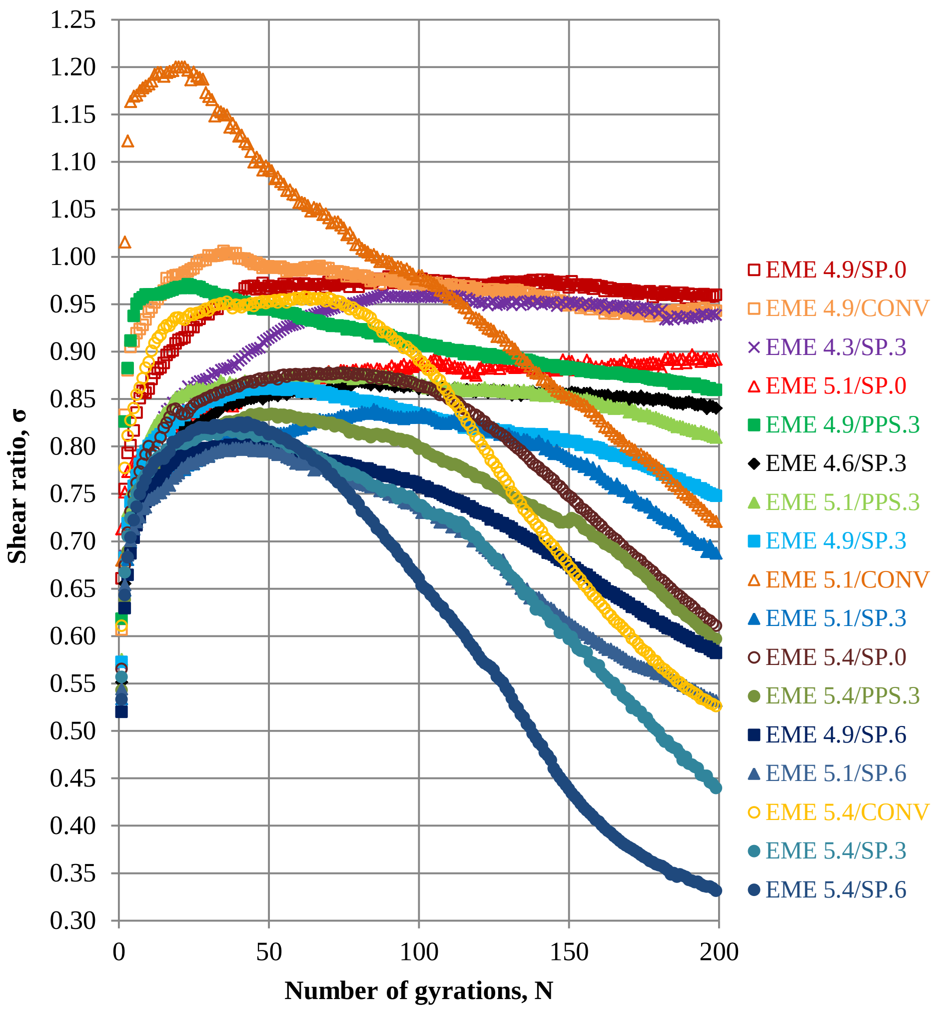

Fig. 3.10 Shear ratio lines of the traditional mixture, of the mixture with SP, of the mixture with PPS and of the conventional mixture

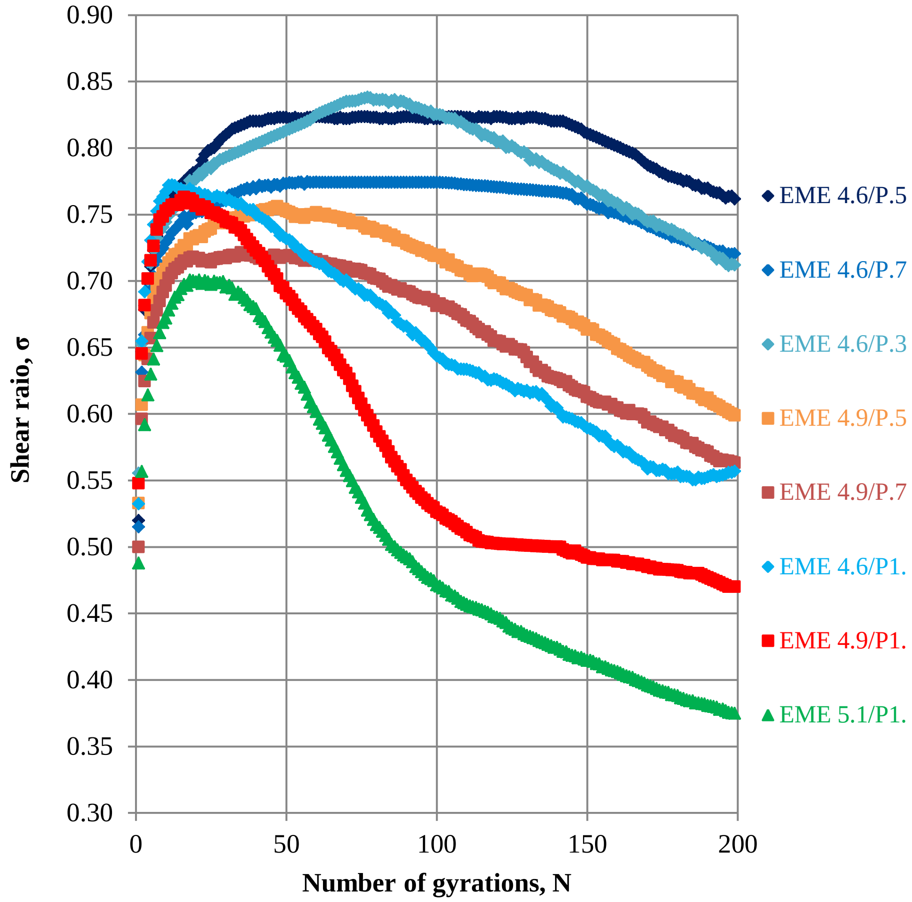

Fig. 3.11 Shear ratio lines of the mixture with waste plastics

The shear ratio lines in Fig. 3.10 show that the specimens with 0.3% of SP and 4.3 or 4.6% of bitumen, and specimens with 0% of SP but 4.9 or 5.1 of bitumen, mobilize shear ratio values that are maintained constant during the design life years. The same can be said for mixtures with 0.3% of PPS and 4.9 or 5.1% of bitumen and for the conventional mixture with 4.9% of bitumen. Regarding the mixture with waste plastics in Fig. 3.11 the specimens significant of an optimal content are those containing 0.5 or 0.7% of plastics and 4.6% of bitumen.

By contrast, a slight excess of bitumen and a slight excess of additive cause a fall of the shear ratio and therefore the content is not optimal.

Regarding voids, Table 3.8 shows the average values of specimens prepared with the gyratory compactor that, considering for example the requirements on the air voids at 10, 100 and 190 rpm, reported in Table 3.9 (Anas S.p.A, 2009), confirm the results seen in Figures 3.10-3.11.

Mixture	b	p	Number of gyrations		
	%	%	10	100	190
EME 4.3/SP.3	4.3	0.3	12.6	4.7	2.8
EME 4.6/SP.3	4.6	0.3	12.1	4.4	2.6
EME 4.9/SP.0	4.9	0.0	13.0	5.1	3.2
EME 4.9/SP.3	4.9	0.3	11.1	2.7	1.0
EME 4.9/SP.6	4.9	0.6	10.2	2.0	0.7
EME 5.1/SP.0	5.1	0.0	11.2	3.8	2.0
EME 5.1/SP.3	5.1	0.3	10.8	2.4	0.8
EME 5.1/SP.6	5.1	0.6	10.2	1.9	0.6
EME 5.4/SP.0	5.4	0.0	9.9	1.7	0.4
EME 5.4/SP.3	5.4	0.3	9.6	1.4	0.3
EME 5.4/SP.6	5.4	0.6	9.5	1.0	0.3
EME 4.6/P.3	4.6	0.3	10.7	2.6	1.2
EME 4.6/P.5	4.6	0.5	11.3	3.3	1.8
EME 4.6/P.7	4.6	0.7	11.6	3.3	1.8
EME 4.6/P1	4.6	1.0	9.3	1.9	1.1
EME 4.9/P.5	4.9	0.5	10.4	2.4	1.0
EME 4.9/P.7	4.9	0.7	10.5	2.6	1.3
EME 4.9/P1	4.9	1.0	8.8	1.6	0.8
EME 5.1/P1	5.1	1.0	8.5	1.6	0.8
EME 4.9/CONV	4.9	0.0	12.5	4.2	2.4
EME 5.1/CONV	5.1	0.0	10.0	1.8	0.4
EME 5.4/CONV	5.4	0.0	10.4	1.2	0.2
EME 4.9/PPS.3	4.9	0.3	11.1	2.6	0.9
EME 5.1/PPS.3	5.1	0.3	11.0	2.4	0.7
EME 5.4/PPS.3	5.4	0.3	10.0	1.5	0.1

Tab. 3.8 Average values of the air voids of the specimens at 10, 100 e 190 gyrations

Number of gyrations	% voids
10	11÷15
100	3÷6
190	≥2

Tab. 3.9 Air void values with variation in the number of gyrations

3.3.2 Permanent deformation resistance

Based on the results of the formulation tests, the wheel-tracking test was carried out on the mixtures studied considering two percentages of bitumen for each percentage of polymer chosen (except for the mixture with waste plastics where only one percentage of bitumen was considered), making overall two slabs for each mixture, with dimensions 305 x 305 x 50 mm and air void content v = 4.5 %. The average values are reported in Figure 3.12.

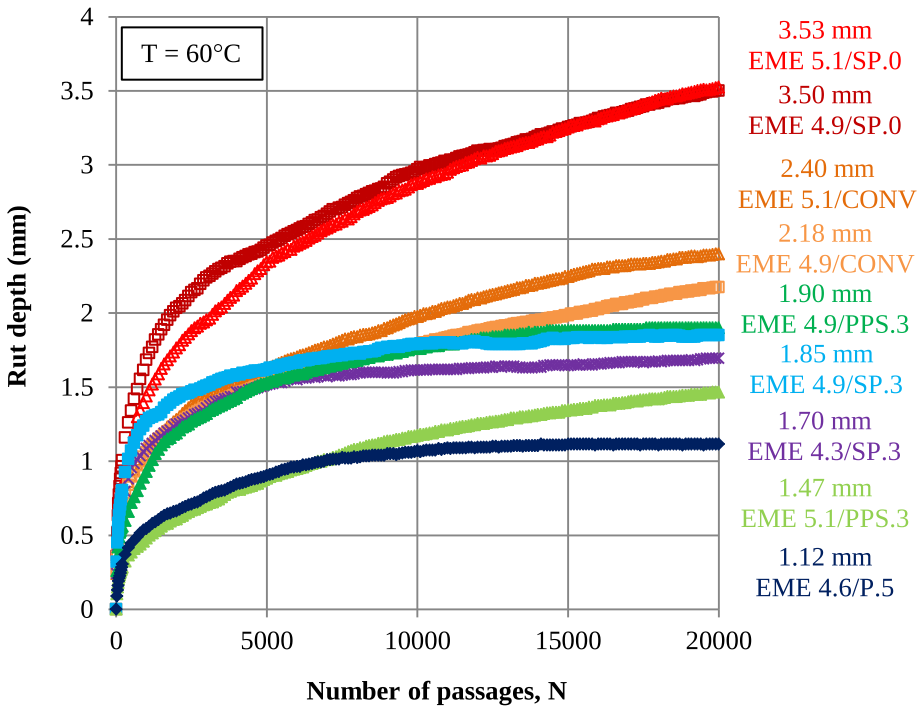

Fig. 3.12 Trend of rut depth in the wheel-tracking test

In Fig. 3.12 it is possible to note that, for the same percentage of polymer, there is no substantial difference regarding rut depth; by contrast, at the same percentage of bitumen, the rut depth values of the mixture with SP are half of those of the mixture without additives. The same can be said more or less for the mixture with PPS, while the best result is for the mixture with waste plastics where the rut depth value is reduced by one third compared to the mixture without additives. Regarding the conventional mixture (produced with a harder bitumen, 35/50) the rut depth values are lower than those of the mixture without additives (produced with 50/70 pen grade bitumen), but they are higher than those of the mixture with additives.

The additives used thus appear to be particularly advantageous regarding permanent deformation resistance, since they reduce rutting, increasing the cohesive ability of the bituminous mixture, as already demonstrated by many tests (Iterchimica S.r.l, 2015). A confirmation can be obtained by calculating an additional parameter like the wheel-tracking slope in air (WTS_{air}), that is the average rate at which rut depth increases with the number of passages (see Table 3.10). The fact is that this parameter increases when the percentage of bitumen increases, while it decreases when the percentage of additive increases. Based on the results, this parameter confirms what was said above.

Mixture	WTS_{air} (mm/10^3 cycles)
EME 5.1/SP.0	0.130
EME 4.9/SP.0	0.106
EME 5.1/CONV	0.085
EME 4.9/CONV	0.077
EME 5.1/PPS.3	0.058
EME 4.9/PPS.3	0.028
EME 4.3/SP.3	0.016
EME 4.9/SP.3	0.012
EME 4.6/P.5	0.009

Tab. 3.10 Values of WTS_{air} (wheel-tracking slope in air) of the mixtures studied

3.3.3 Stiffness modulus

The loading configuration adopted in the stiffness modulus tests (as well as in the fatigue tests) was sinusoidal bending on prismatic specimens constrained at the two outer clamps and point loads at the two inner clamps (four-point bending beam test). The deformation was kept constant during the test (controlled strain).

The stiffness modulus test was carried out, according to EN 12697-26 annex B, on beams with dimensions 400 x 50 x 45 mm, before each fatigue test with the sole difference that the test did not finish when the beams broke or when the value was half its initial value (at 100th cycle), but was only carried out 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. The complex modulus test and the fatigue test were carried out on optimized mixtures, that is the ones with the best results after formulation tests and wheel-tracking test, using a four-point bending apparatus and the GCTS CATS software.
The stiffness modulus test parameters are reported in Table 3.11 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; besides, the mixture with waste plastics has the highest stiffness modulus values at the same temperature and frequency conditions as the other mixtures. The conventional mixture produced with the 35/50 pen grade bitumen confirms higher values than the mixture with 50/70 pen grade bitumen, due to the presence of harder bitumen.

Mixture	$\vert E^* \vert$ (MPa)			ϕ (Deg)		
	Frequency (Hz)			Frequency (Hz)		
	1	10	30	1	10	30
EME 4.9/SP.0	3692	7282	8756	36	24	23
EME 4.3/SP.3	4168	7528	9188	30	20	18
EME 5.1/PPS.3	4240	7874	9704	31	22	19
EME 4.9/CONV	5451	9378	11138	28	19	17
EME 4.6/P.5	6793	10682	12277	23	15	15

Tab. 3.11 Values of complex modulus and phase angle of the mixtures studied

Finally, the dynamic modulus was also evaluated by means of a triaxial cell. This test consists of applying a sinusoidal axial compression stress on a specimen of asphalt concrete at a given temperature and loading frequency. The resulting recoverable axial strain response of the specimen is measured and used to calculate dynamic modulus. The loading configuration adopted for characterizing the stiffness of the bituminous mixture (dynamic modulus) was direct compression on cylindrical specimens. The tension was kept constant during the test (controlled stress), which was carried out according to the EN 12697-26 standard, annex D.

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 x 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 (Celauro et al, 2010). 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. Figure 3.13 shows the master curves for the mixtures studied with 5% voids.

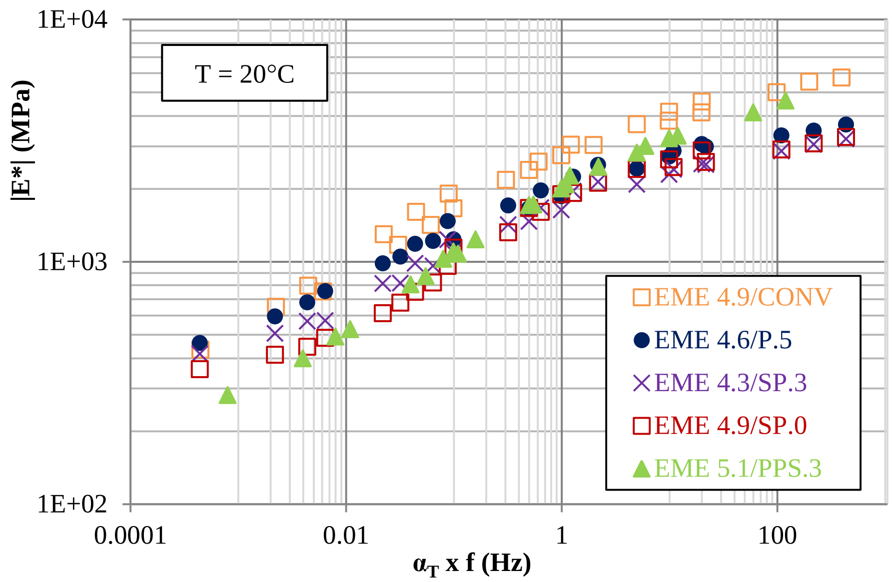

Fig. 3.13 Master curves of the mixtures studied

From Figure 3.13 it can be said what it was reasonable to expect: the dynamic modulus values of the conventional mixture produced with 35/50 pen grade bitumen are higher than those of the mixture produced with 50/70 pen grade bitumen in the whole range of frequencies studied. Regarding the other mixtures it is interesting to note that at low frequencies the mixture with waste plastics provides similar values to the conventional mixture, while at high frequencies it provides similar values to the mixture produced with 50/70 pen grade bitumen; in the case of the mixture with PPS we see just the opposite, since at low frequencies it provides similar values to the mixture produced with 50/70 pen grade bitumen, while at high frequencies it provides similar values to the conventional mixture. Lastly, the mixture with SP provides similar values to the mixture without additives produced with 50/70 pen grade bitumen. It is clear, therefore, that the chemical nature of the polymers used affects the final result.

These test results lead to the consideration that additives can positively influence the mechanical performance of the mixture, that is in terms of stiffness modulus it is possible to obtain the same values as for a mixture produced with hard bitumen by using less hard bitumen and adding polymers.

3.3.4 Fatigue cracking

As already mentioned above, the fatigue behaviour of the optimized mixtures was studied using a four-point bending apparatus and the GCTS CATS software.

The fatigue criterion used was the classical one, referenced as N_{f50}. It corresponds 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}".

The tests were conducted at 20°C and 10 Hz, according to EN 12697-24 annex D, on beams with dimensions 400 x 45 x 50 mm and obtained from a slab of dimensions 400 x 305 x 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) shown in Figure 3.14, i.e. Eq. (3.4).

$$\varepsilon = a \times N^{-b} \tag{3.4}$$

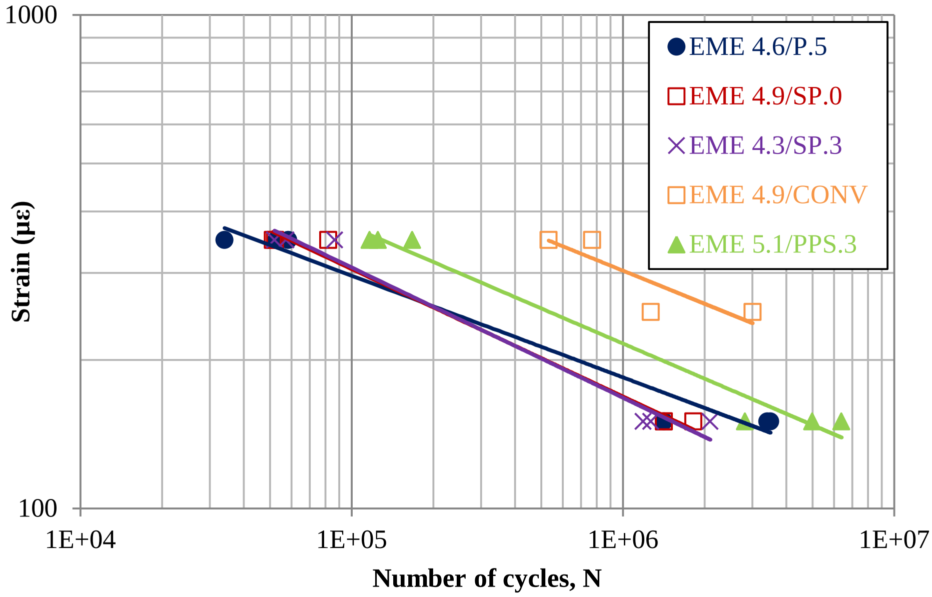

Fig. 3.14 Fatigue lines

For each fatigue line the following parameters were calculated in order to make a judgment on performance:

- "a" is a constant and it depends on the physical and mechanical characteristics of the material, test temperature and frequency;
- slope of the fatigue lines (b);
- coefficient of determination (R^2);
- admissible strain level at $N = 10^6$ loading applications ($\varepsilon_{10}{}^6$) in order to characterize fatigue resistance.

Table 3.12 summarizes these parameters for the mixtures studied.

Mixture	A	B	R^2	$\varepsilon_{10}{}^6$
EME 4.3/SP.3	4532	-0.240	0.9797	164.5
EME 4.9/SP.0	6328	-0.262	0.9828	169.5
EME 4.6/P.5	3115	-0.206	0.9660	180.9
EME 5.1/PPS.3	5651	-0.236	0.9773	216.8
EME 4.9/CONV	6582	-0.223	0.7385	302.2

Tab. 3.12 Fatigue line parameters of the mixtures studied

The fatigue lines have high values of the regression coefficients R^2 (see Tab. 3.12) and this means that the results are only slightly dispersed and very reproducible.

By comparing the fatigue lines, it can be noticed that the mixtures with additives offer fatigue performance that is similar to (as in the case of the mixture with SP) or better than (as in the case of the mixtures with PPS or waste plastics) that of the mixture without additives, produced with 50/70 pen grade bitumen. Nevertheless, such mixtures do not reach the high performance levels of the conventional mixtures in terms of fatigue resistance.

In any case, all admissible strain values ε_6 (>130 μstrain) can be considered satisfactory for road paving applications, showing that the addition of polymers is a viable solution.

3.4 Conclusions

Limited to the materials studied, the laboratory tests can be considered adequate to indicate the possibility of using the additives in order to improve mechanical performances of bituminous mixtures produced with component of average quality.

The optimized mixtures show good stability values and compaction and the positive influence that the additives studied have on these mixtures regarding permanent deformation resistance (at the same bitumen content, mixtures with additives are more resistant than mixtures without additives, produced with 50/70 or 35/50 pen grade bitumen).

Moreover, the presence of the additives allows optimization of mixtures with lower binder contents and also makes it possible to obtain higher stiffness modulus values than traditional mixtures without additives as well as the values of the high modulus asphalt mixtures, represented by the conventional mixture. Regarding fatigue resistance it can be said that mixtures with additives do not offer the same high performance as the conventional mixture; however, their presence leads to similar or better results than those offered by a mixture without additives produced with 50/70 pen grade bitumen.

A possible future development is a thorough statistical study considering several kinds of bitumen and locally available aggregate sources, as locally available in order to highlight performance sensitivity to aggregate-bitumen combinations, as well as considering many other types of polymer, particularly wastes, since they can definitely be an economic and ecological alternative for paving works.

Chapter 4

Case 3

4.1 General

Case 3 is an experimental study concerning plastic additives used in the surface, binder and base courses of the access pavement to the new apron of Palermo International Airport.

The focus was on comparison between data obtained from laboratory tests and data obtained from field tests.

For this purpose, the stiffness modulus was evaluated through triaxial cell tests at the University of Palermo's Road Materials Laboratory and it was compared to the stiffness modulus values obtained using BAKFAA, a software program made available by the Federal Aviation Administration (FAA), in which deflection values obtained by Heavy Weight Deflectometer (HWD) tests were used.

The test results confirm the high modulus values obtained using additives in asphalt mixtures and thus justify their use in settings such as airports, where the pavements are subject to high loads, which lead to gradual surface degradation.

4.2 Materials

The asphalt mixtures that are the object of this case study are mixtures with plastic additives used in the surface, binder and base courses of the access pavement to new apron of Palermo Airport, not yet open to air traffic.

The materials used to produce these mixtures will now be presented.

4.2.1 Aggregates

The stone aggregates used in the mixtures came from crushed limestone (supplied by I.s.a.p. S.r.l.), whose composition and physical and mechanical properties are given respectively in Table 4.1 and Table 4.2.

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

Tab. 4.1 Composition of the aggregates' available fractions

Characteristics	Fractions					
	25/30	20/25	10/15	5/10	0/6	filler
Bulk specific weight, g/cm^3 (EN 1097-7)						2.85
Apparent specific weight, g/cm^3 (EN 1097-6)	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 (CNR 65)	0.86	0.78	0.80	0.73	0.70	
Absorption coefficient (EN 1097-6)	0.51	0.48	0.48			

Tab. 4.2 Physical and mechanical characteristics of the available aggregates

The mix grading curves were obtained for the composition of the fractions provided by implantation as shown in Figures 4.1-4.3.

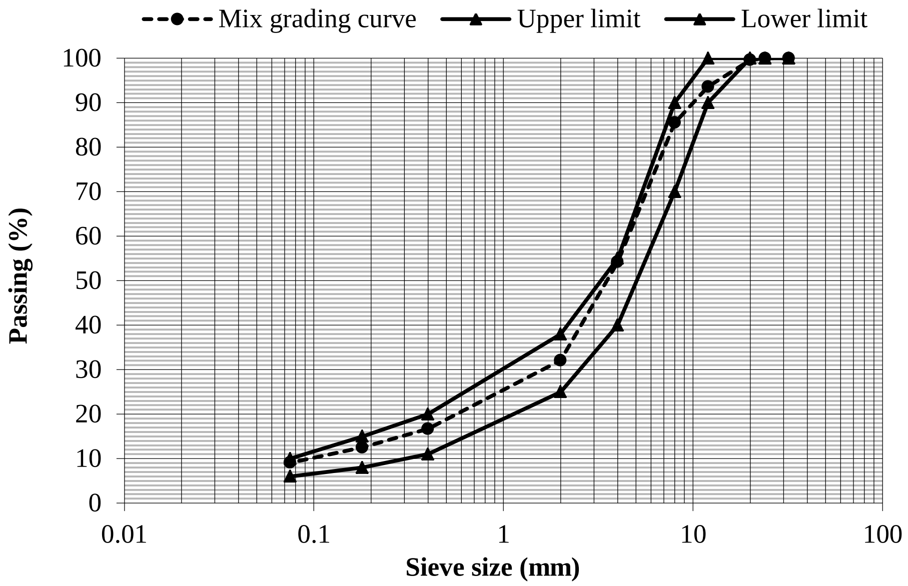

Fig. 4.1 Mix grading curve for the surface course

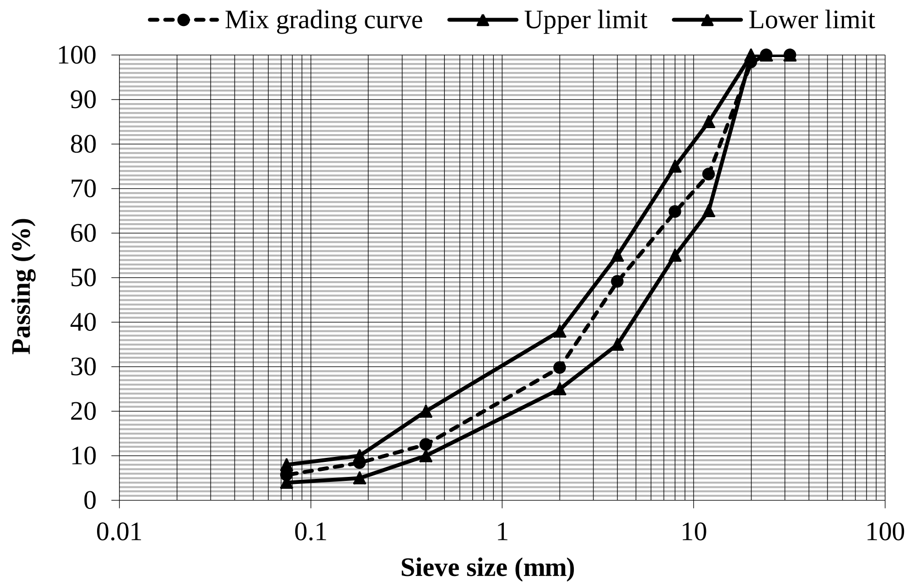

Fig. 4.2 Mix grading curve for the binder course

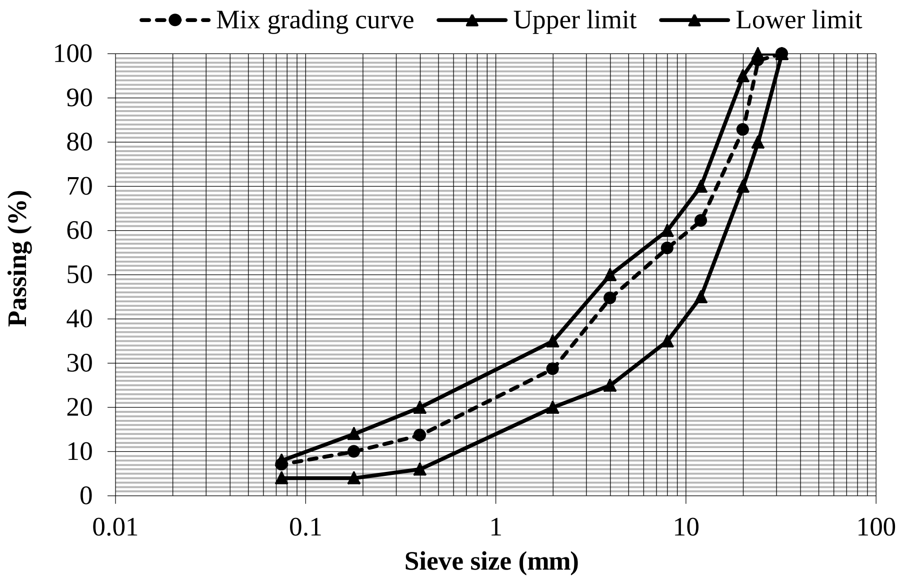

Fig. 4.3 Mix grading curve for the base course

4.2.2 Bitumens

The bitumen used in this research was neat bitumen 50/70; its characteristics are reported in Table 4.3.

Characteristic	Unit	Value	Standard
Specific weight at 25°C	g/cm^3	1.033	EN 3838
Penetration at 25°C	dmm	68	EN 1426
Ring and Ball Softening Point	°C	50.5	EN 1427
Penetration Index		-0.21	EN 12591
Fraas Temperature	°C	-12	EN 12593
Ductility at 25°C	cm	> 100	ASTM D113
Viscosity at 60 °C	Pa·s	270	EN 13302
Viscosity at 160 °C	Pa·s	0.11	EN 13302
After RTFOT:			
Change in mass	%	0.19	EN 12607-1
Penetration at 25°C	dmm	44	EN 1426
Ring and Ball Softening Point	°C	54	EN 1427
Viscosity at 60 °C	Pa·s	668	EN 13302

Tab. 4.3 Characteristics of 50/70 pen grade bitumen

4.2.3 Additives

The additive used (SP) was already mentioned in Chapter 3; its properties are listed in section 3.2.3.

4.3 Methods, results and discussion

The experimental study was developed on the following mixtures:

- SURF 0/20 (50/70+SP);
- BINDER 0/20 (50/70+SP);
- BASE 0/25 (50/70+SP).

The mix design was achieved by carrying out the Marshall test with 3 percentages of bitumen and CE equal to 75 blows per face, according to the EN 12697-34 standard. This test was carried out by Iterchimica S.r.l.

These mixtures were used in the surface, binder and base courses of the access pavement to the new apron of Palermo Airport.

Laboratory tests on optimized mixtures were performed in order to assess the stiffness modulus, according to the EN 12697-26 standard, annexes B and D.

Finally, a Heavy Weight Deflectometer (HWD) Test conducted with the Dynatest device of the University of Messina at Palermo Airport made it possible to obtain the deflection values from the pavement. These values were then entered into the software program BAKFAA for a back-calculation analysis in order to compare the laboratory results with the field data.

4.3.1 Mix design

The mix design was performed with the Marshall Method, according to the EN 12697-34 standard, using three bitumen contents and one SP content for different courses (b'_1 = 4.8%, b'_2 = 5.6%, b'_3 = 6.5%, by mass of aggregates, and SP = 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 SP = 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 SP = 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 air void content was determined according to the EN 12697-8 standard, while the calculation of the apparent specific weight (γ_{app}) was performed according to the EN 12697-6 standard. The results are reported in Tables 4.4-4.6, where S is the Marshall Stability, F is the Marshall Flow, which is the corresponding displacement, and R the Marshall Ratio S/F.

Considering the requirements in the Italian Specifications regarding the physical and mechanical characteristics, such as for example the ones in Table 4.7 (MIT, 2002), Marshall test results are also reported in Figures 4.4-4.6.

b (%)	S (KN)	F (mm)	R (KN/mm)	v (%)	γ_{app} (g/cm^3)
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

Tab. 4.4 Marshall test results for the surface course

b (%)	S (KN)	F (mm)	R (KN/mm)	v (%)	γ_{app} (g/cm^3)
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

Tab. 4.5 Marshall test results for the binder course

b (%)	S (KN)	F (mm)	R (KN/mm)	v (%)	γ_{app} (g/cm^3)
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

Tab. 4.6 Marshall test results for the base course

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

Tab. 4.7 Marshall test results in accordance with MIT Specification

For the surface course, the specimens significant of an optimal content are those containing 8% (by mass of bitumen) of SP and 5.6% (by mass of aggregates) of bitumen; for the binder course, the ones containing 5.5% (by mass of bitumen) of SP and 4.9% (by mass of aggregates) of bitumen, and finally, for the base course, the ones containing 5.5% (by mass of bitumen) of SP and 4.1% (by mass of aggregates) of bitumen.

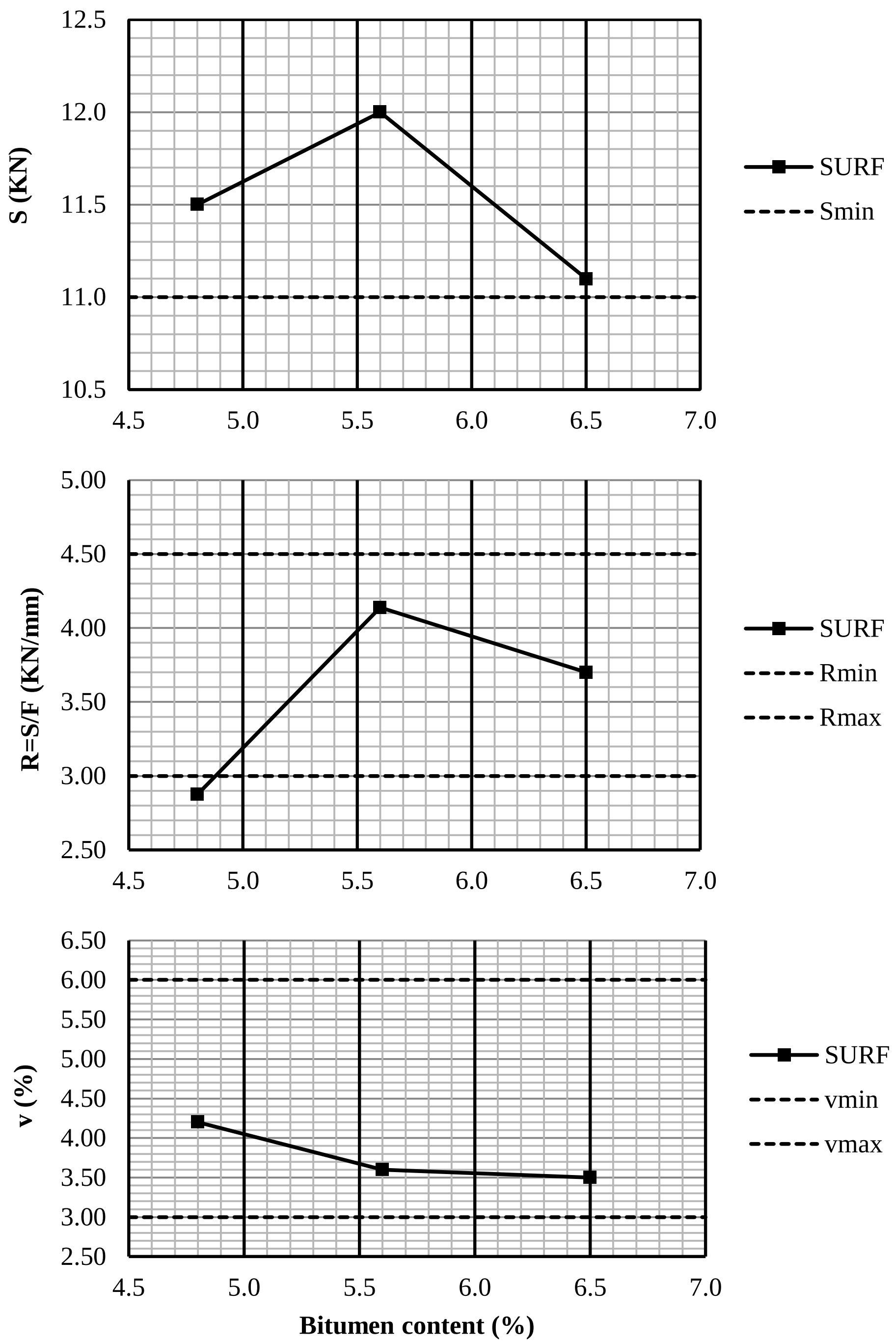

Fig. 4.4 Marshall values at different bitumen contents for the surface course

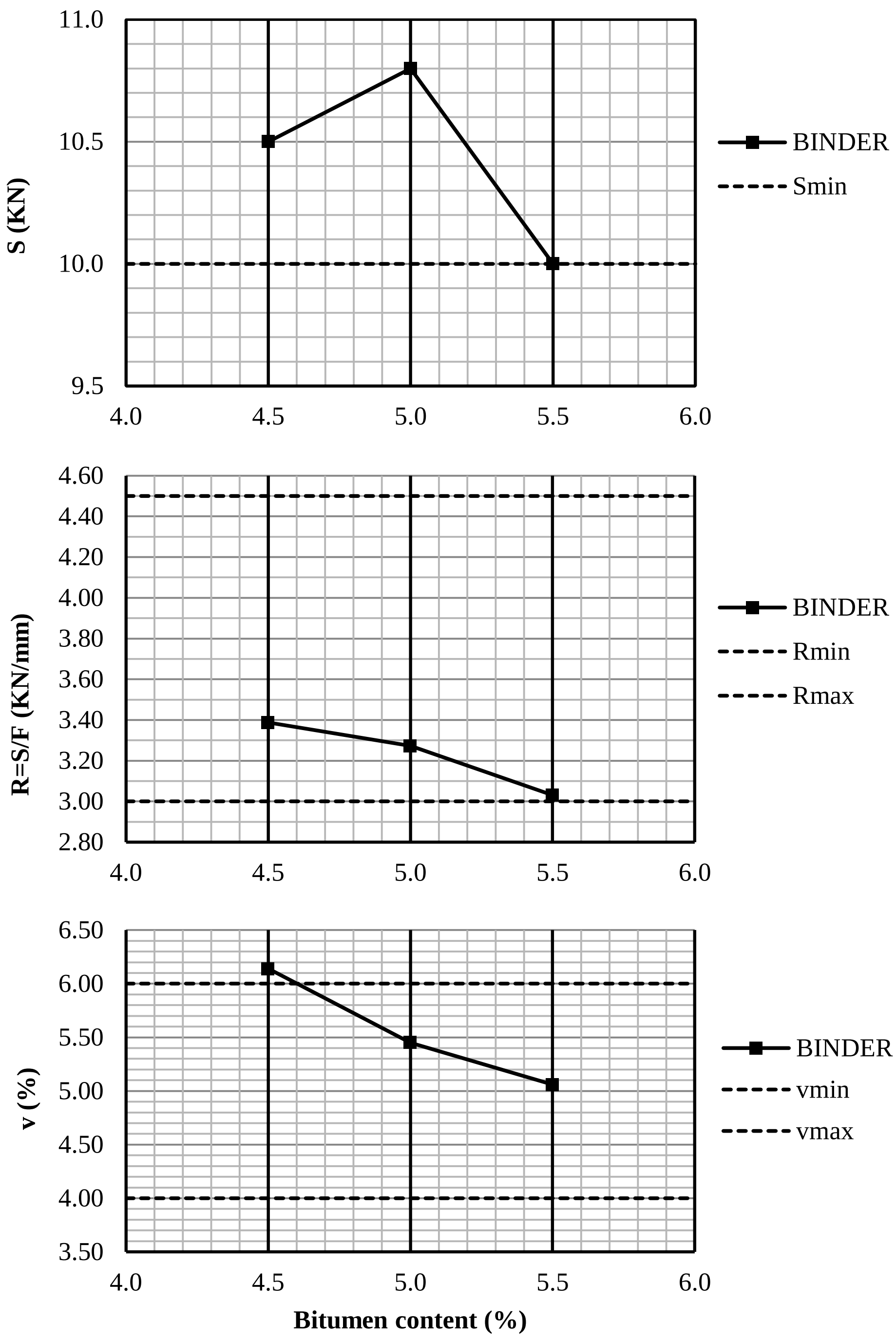

Fig. 4.5 Marshall values at different bitumen contents for the binder course

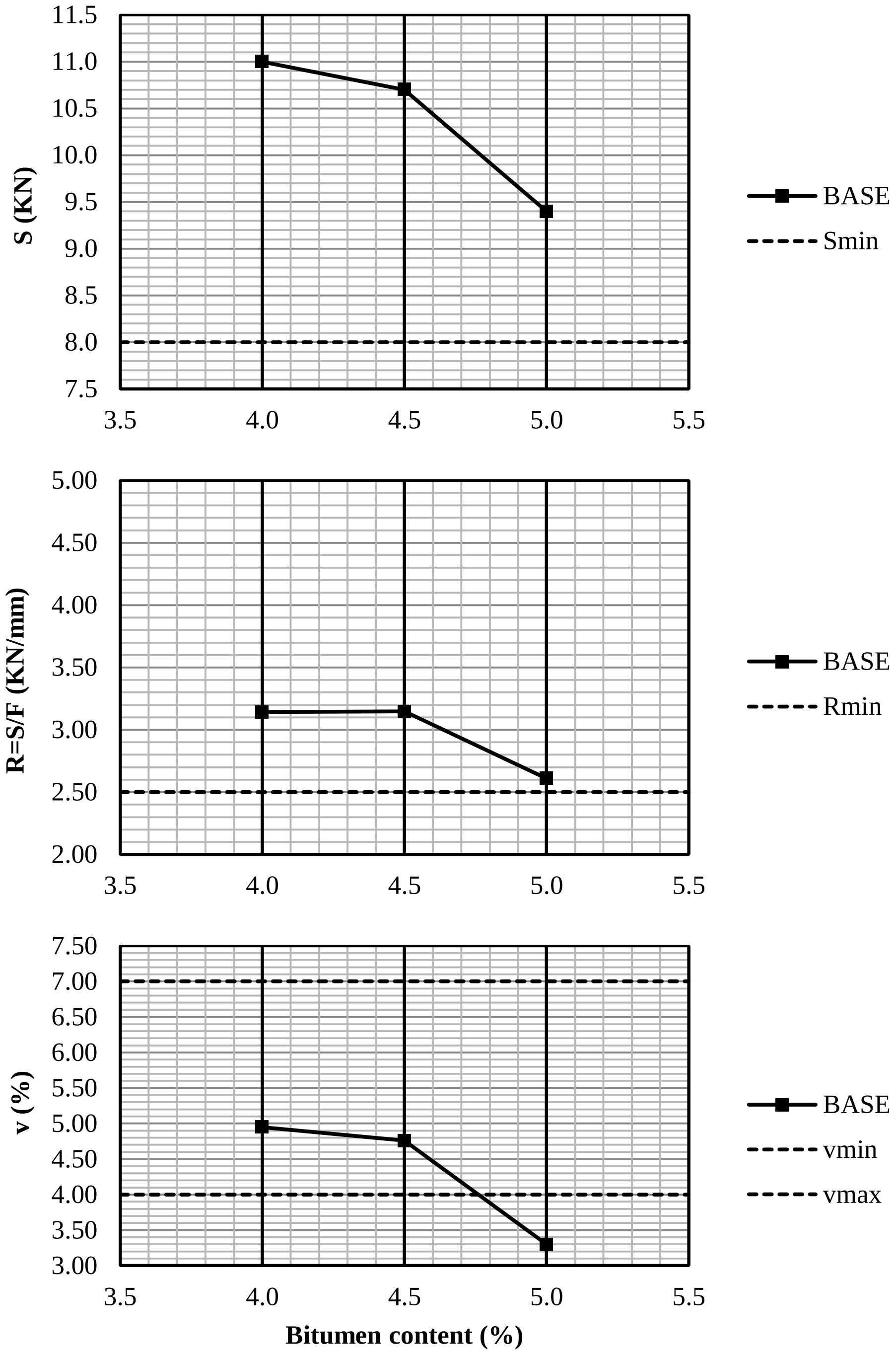

Fig. 4.6 Marshall values at different bitumen contents for the base course

4.3.2 Stiffness modulus

The stiffness modulus test was carried out on optimized mixtures, according to EN 12697-26 annex B (four-point bending test on prismatic beams), on beams with dimensions 400 x 50 x 45 mm for the surface course and 400 x 50 x 50 mm for the binder course.

The deformation was 25 $\mu\varepsilon$, 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.

This test was carried out only for the surface and binder courses, because in the base course the maximum diameter size (D_{max}) of the mix grading was inconsistent with the testing equipment. The stiffness modulus test parameters are reported in Table 4.8.

Mixture	$\lvert E^* \rvert$ (MPa)			ϕ (Deg)		
	Frequency (Hz)			Frequency (Hz)		
	1	10	30	1	10	30
SURF	5193	8256	9461	23	16	16
BINDER	7037	11250	11903	21	13	13

Tab. 4.8 Values of complex modulus and phase angle for the surface and binder courses

The results 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; besides, the mixture for the binder course has the higher stiffness modulus values than those of the mixture for the surface course at the same temperature and frequency conditions.

In this connection, the highest modulus values are required in the lower layers, while for the surface courses the highest values are required regarding functional properties.

High values obtained for these mixtures confirm what was said previously regarding the advantage of using polymeric additives in asphalt mixtures.

Finally, the stiffness modulus was also evaluated by means of a triaxial cell, whose loading configuration was described in the previous chapter.

The tests were carried out according to the EN 12697-26 standard, annex D, and 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 x 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, at a reference temperature of 30°C for each bituminous mixture. Figures 4.7-4.9 show the master curves for the mixtures studied with 4.5% voids.

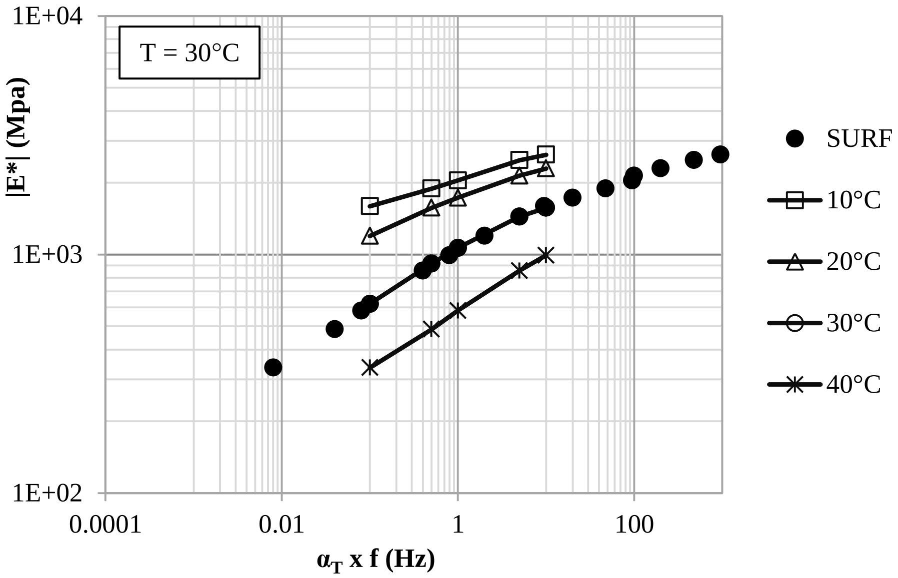

Fig. 4.7 Complex modulus master curve for the surface course

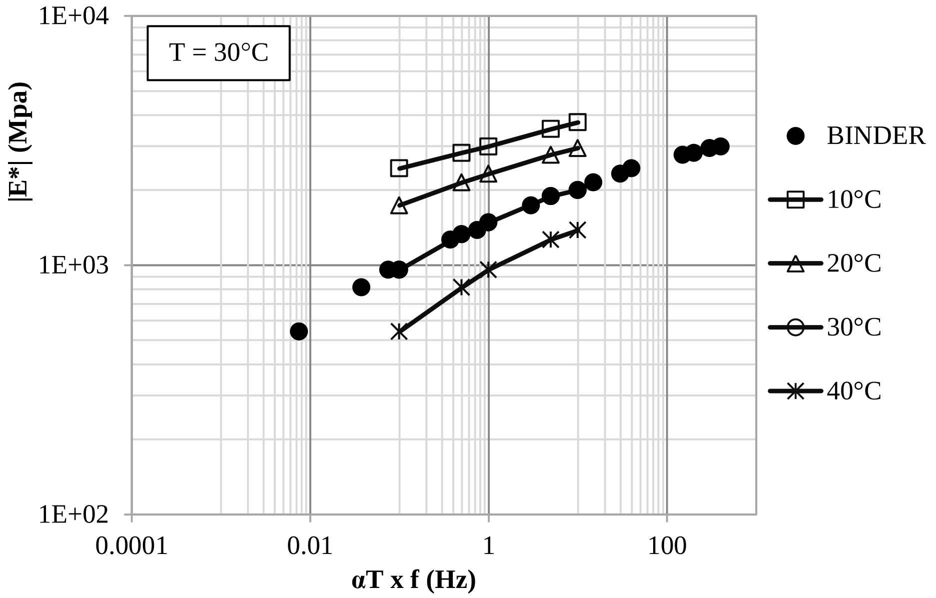

Fig. 4.8 Complex modulus master curve for the binder course

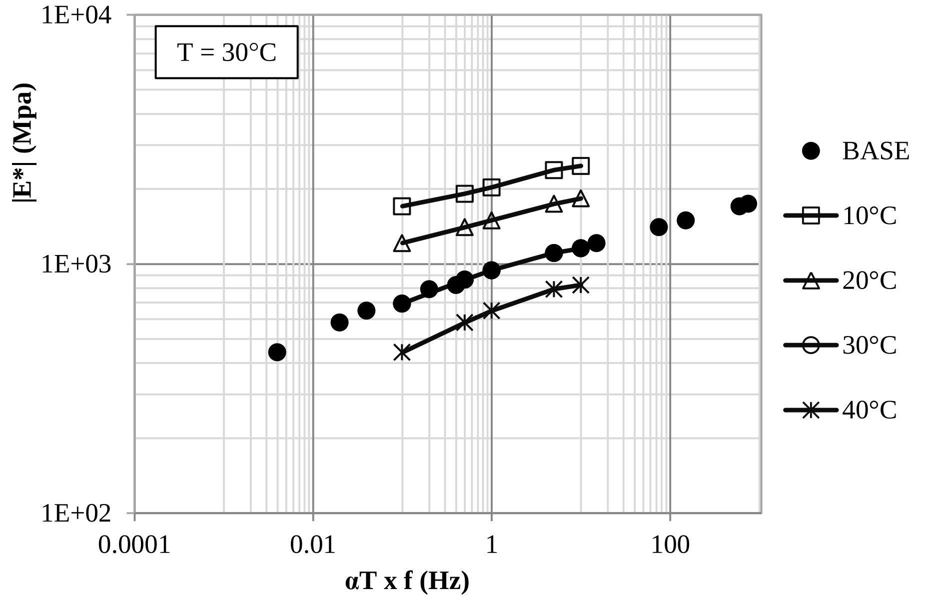

Fig. 4.9 Complex modulus master curve for the base course

Master curves show stiffness modulus values in line with the results of the other mixture with additives, seen in the previous chapter.

4.3.3 HWD test

The field test used was a Heavy Weight Deflectometer (HWD) test. This test was used to obtain the deflection data from the existing pavement, that is the one of access to the new apron of Palermo International Airport.

The purpose of the HWD test was to analyze the deflection basin to determine the overall structural strength and back-calculating elastic modulus of each layer. A HWD is a falling weight deflectometer (FWD) that uses higher loads, used primarily for testing airports pavements.

In an FWD/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 (Amadore et al., 2014). The deflection of a pavement represents an overall "system response" of the pavement layers to an applied load. A conventional Asphalt Concrete (AC) pavement is typically made up of three layers: a surface layer paved with AC mix, a base or/and subbase layer made up of crushed stone, and a subgrade layer made up of natural soil. When a wheel load is applied on an AC pavement, the pavement layers deflect almost vertically to form a basin.

The deflected shape of the basin, as shown in Figure 4.10, is predominantly a function of the thickness of the pavement layers, the moduli of the individual layers, and the magnitude of the load.

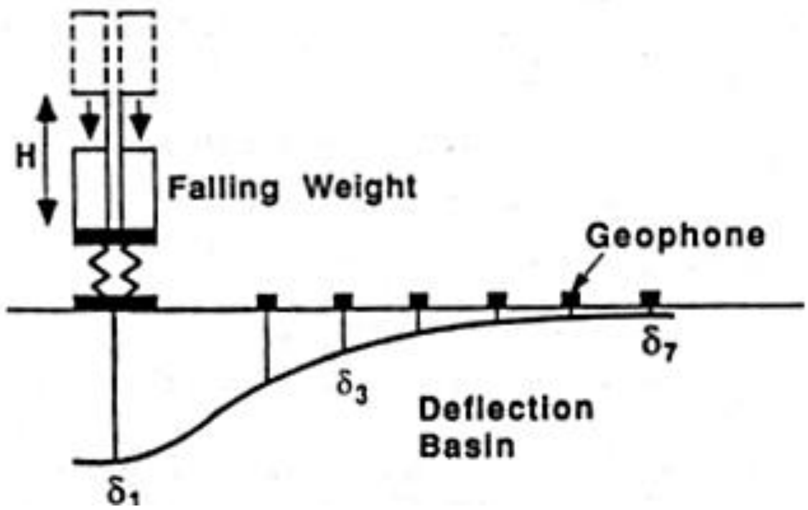

Fig. 4.10 Schematic of FWD/HWD load-geophone configuration (Gopalakrishnan & Thompson, 2004)

In Figure 4.11 there is an example with nine geophones aligned, which make it possible to obtain the deflection value recorded directly under the loading plate.

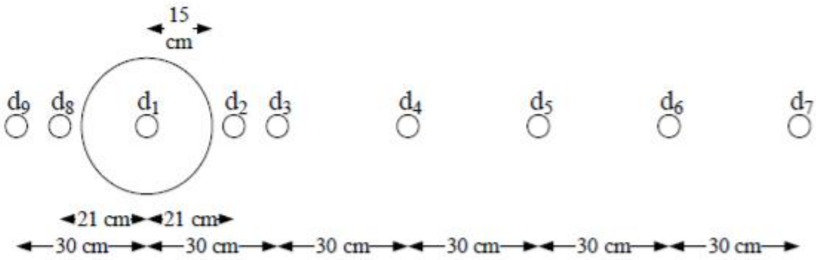

Fig. 4.11 Layout of Geophones (Chou et al., 2007)

Regarding load impact systems there are two different types: single-mass and double-mass. In a single-mass system (the system of testing equipment used) a weight is dropped onto a single buffer connected to a load plate, which rests on the surface being tested. The load force is transferred through the plate, and the plate creates a deflection that simulates a wheel load. The HWD test tries to replicate the force history and deflection magnitudes of a moving aircraft tyre (Gopalakrishnan & Thompson, 2004).

The HWD test was performed at Palermo International Airport in collaboration with the University of Messina using Dynatest testing equipment in their possession, as shown in Figure 4.12.

Fig. 4.12 Dynatest testing equipment

To understand the present structural condition of the pavement in question (a scheme is shown in Figure 4.13), an HWD test was conducted that evaluates the pavement structural strength. The maximum load was around 240 KN.

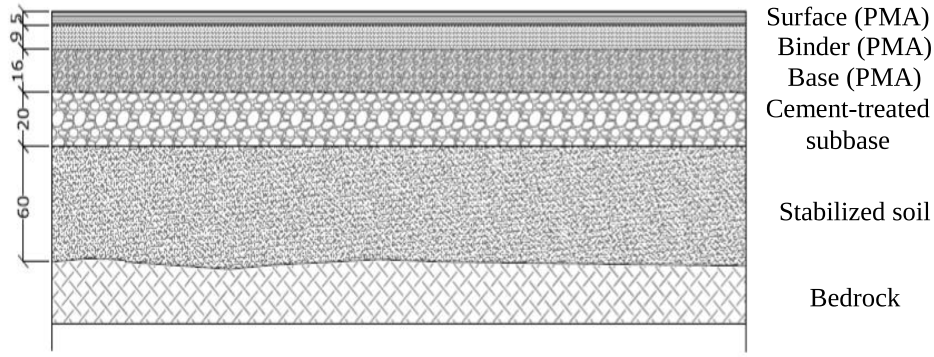

Fig. 4.13 Scheme of the pavement in question

The pavement in question is a semi-rigid pavement, that is 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 SP), a cement treated subbase, a cement stabilized soil and the bedrock formed by calcarenite stones.

The test plan and sampling locations are shown in Figure 4.14. Three drops for every station (at every alignment there were four stations 30 m apart) were carried out on both alignments, as highlighted in the red box.

Data acquisition was followed by the process of calculating the elastic moduli of individual layers in a multilayer system based on surface deflections. This process is known as “back-calculation”.

Back-calculation is one of the most common methods used to analyze the deflection basin, which is collected from HWD. First initial moduli are assumed, then surface deflections are calculated, and finally the moduli are adjusted in an iterative fashion to converge on the measured deflections (Amadore et al., 2014).

As there are no closed-form solutions to accomplish this task, a mathematical model of the pavement system (called a forward model) is constructed and used to compute theoretical surface deflections with assumed initial layer moduli values at the appropriate HWD loads. Through a series of iterations, the layer moduli are changed, and the calculated deflections are then compared to the measured deflections until a match is obtained within the tolerance limit (Gopalakrishnan & Thompson, 2004).

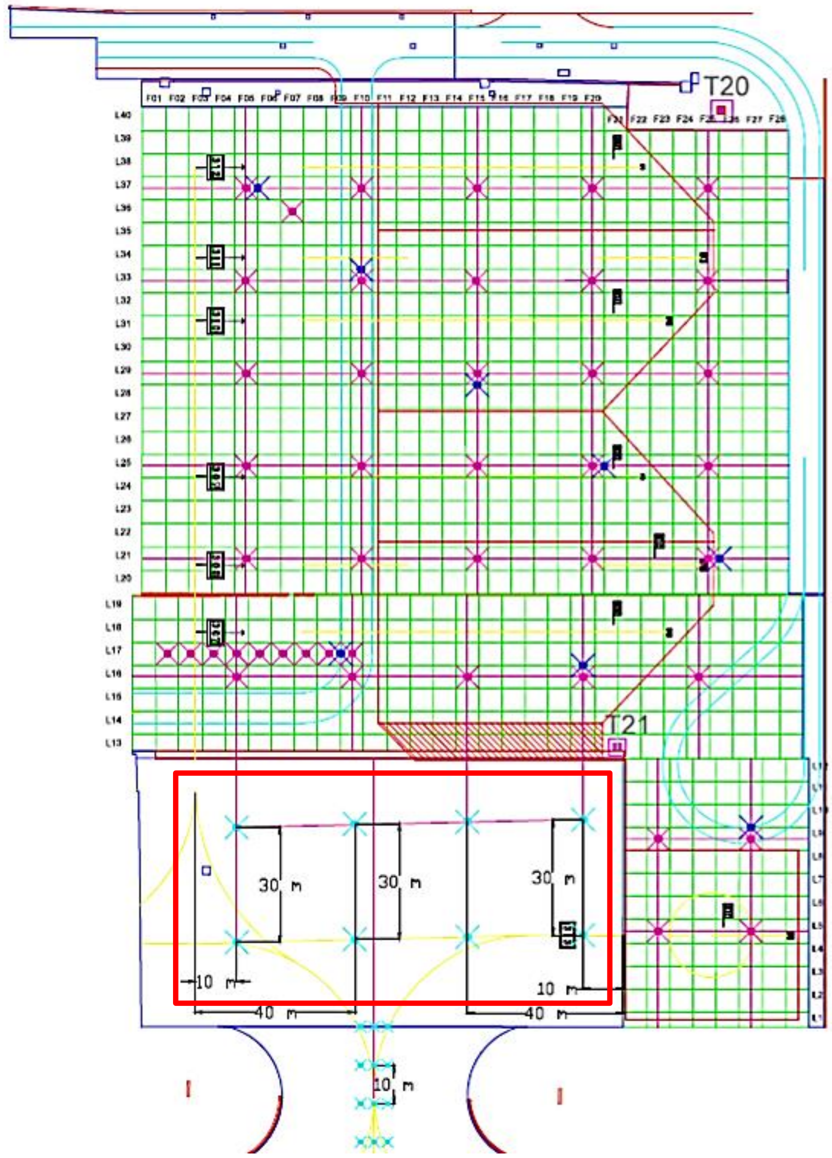

Fig. 4.14 Test plan and sampling locations

This process is computationally intensive although quick on modern computers. There are quite a few well-known back-calculation programs available from which a suitable software for the case study was selected. In this study the back-calculation program used was BAKFAA.

Then laboratory tests were conducted for each bituminous material layer in order to acquire the material properties for verifying the back-calculation results.

4.3.3.1 BAKFAA

BAKFAA is a software program made available by the Federal Aviation Administration (FAA). It performs back-calculation of pavement layer modulus values using the FAA layered elastic analysis program called LEAF 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 FWD/HWD and vertical pavement surface deflections computed with the layered elastic program.

As reported in the guide, BAKFAA requires a minimum number of inputs to adequately characterize the material properties of each layer in a pavement structure and its response to loading:

- layer number (from 1 up to a maximum of 10 layers);
- Young's Modulus of Elasticity, that is a constant ratio of stress and strain, such as the values supplied by the FAA in Advisory Circular AC 150/5370-11B in Table 4.9;
- Poisson's ratio, that is the ratio of transverse to longitudinal strains of a loaded specimen (from 0 to 0.5), such as the values supplied by the FAA in Advisory Circular AC 150/5370-11B in Table 4.10;
- interface parameter, which represents the bond between two pavement layers (from 0 = no bond to 1 = 100% bonding);
- thickness of each pavement layer;
- layer changeable (yes/no), which represents whether the associated layer will allow the modulus seed value to be computed during the back-calculation process or not.

Material	Modulus values (MPa)
Asphalt Concrete	500÷14000
Portland Cement Concrete	7000÷60000
Lean-concrete Base	7000÷20000
Asphalt-treated Base	700÷10000
Cement-treated Base	1400÷14000
Granular Base	70÷350
Granular Subbase or Soil	30÷200
Stabilized Soil	70÷1400
Cohesive Soil	20÷170

Tab. 4.9 Typical Modulus values for paving materials

Material	Poisson’s ratio values
Asphalt Concrete or Asphalt-treated Base	0.25÷0.40
Portland Cement Concrete	0.10÷0.20
Lean-concrete Base or Cement-treated Base	0.15÷0.25
Granular Base, Subbase, or Soil	0.20÷0.40
Stabilized Soil	0.15÷0.30
Cohesive Soil	0.30÷0.45

Tab. 4.10 Typical Poisson’s ratio values for paving materials

The input values used were the ones reported in Table 4.11.

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

Tab. 4.11 Input values used for the pavement in question

Then it is necessary to insert the sensor (geophone) offset, which is measured from the point of loading, and measured deflections (D_i) during HWD testing (the values are reported in Tables 4.12-4.13) and these values are static within the BAKFAA software. It is possible to insert only 7 deflection values, and therefore they were chosen considering offsets (distances from the axis of loading) of 0, 200, 300, 450, 600, 1200, 1500 mm (excluding offsets of 900 e 1800 mm). With each iteration of the back-calculation cycle, the Young's Modulus of each changeable layer is adjusted in order to compute a new set of calculated deflections. Then a value of root mean square (RMS) of the differences between measured and calculated deflections is compared with a process tolerance value. Then each text box representing a calculated deflection value at a specific sensor location is updated. The calculation process terminates when the RMS value is less than or equal to a pre-determined tolerance value.

Station	Drop	D1	D2	D3	D4	D5	D6	D7	D8	D9
1	1	297.90	40.60	19.90	14.20	11.30	9.00	6.20	4.00	2.10
1	2	334.70	61.00	30.20	21.80	18.70	13.10	9.50	6.30	3.90
1	3	675.90	135.60	70.10	51.60	44.20	31.10	21.10	14.50	10.20
2	4	159.90	30.40	16.90	10.20	8.20	5.90	4.30	2.50	0.30
2	5	230.20	49.50	26.20	17.60	13.60	9.80	7.30	4.20	1.60
2	6	513.60	122.60	65.30	44.30	34.90	23.90	16.60	10.10	7.10
3	7	158.40	45.60	33.80	28.00	24.80	19.00	15.00	11.40	9.10
3	8	226.00	71.90	53.60	44.10	38.80	29.60	23.00	17.20	12.60
3	9	501.00	177.90	135.40	113.10	100.00	76.40	58.30	43.20	33.50
4	10	152.40	48.70	35.50	29.00	25.90	20.20	17.00	16.20	16.70
4	11	230.30	78.60	57.40	45.90	41.00	33.00	26.50	22.00	18.90
4	12	526.80	188.30	139.10	112.40	98.40	78.60	61.50	49.50	41.10

Tab. 4.12 Measured deflections for the alignment 1

Station	Drop	D1	D2	D3	D4	D5	D6	D7	D8	D9
1	1	253.40	48.00	27.40	20.90	19.70	14.10	9.30	7.00	6.80
1	2	347.10	73.90	44.10	32.90	29.10	22.10	15.70	10.40	5.20
1	3	818.80	179.80	111.80	83.40	73.20	54.00	37.30	25.60	15.20
2	4	188.80	60.30	41.70	30.50	26.00	18.50	14.00	8.80	6.80
2	5	273.00	96.30	66.30	48.60	40.60	29.80	21.80	14.40	9.80
2	6	631.90	249.40	178.80	132.50	110.60	79.10	56.80	38.40	26.30
3	7	189.60	62.30	44.90	34.60	30.40	23.90	20.30	14.00	10.00
3	8	276.40	98.90	71.40	54.90	47.90	38.10	31.30	23.30	18.60
3	9	646.70	258.50	192.00	145.30	123.30	96.00	76.20	57.90	46.50
4	10	166.80	55.40	43.20	35.00	31.50	23.60	19.20	14.00	12.00
4	11	241.50	88.80	67.50	56.50	50.00	39.40	30.90	23.60	20.70
4	12	534.80	218.30	167.60	138.70	120.90	94.20	72.30	54.40	45.10

Tab. 4.13 Measured deflections for the alignment 2

After calculating the deflections, it is necessary to select load and run LEAF (Layered Elastic Analysis Program in Forward Mode). The purpose of this is to compute the pavement response for various aircraft landing gear geometries. To compute the stresses, strains and deflections at any point in a pavement structure resulting from the application of a surface load BAKFAA uses a layered elastic mechanistic model.

Layered elastic models assume that each pavement structural layer is homogeneous, isotropic, and linearly elastic. In other words, it is the same everywhere and will rebound to its original form once the load is removed. It works based on the Boussinesq mathematical model and, thus, requires some basic assumptions, which are the following:

- the pavement layers extend infinitely in the horizontal direction;
- the bottom layer (usually the subgrade) extends infinitely downward;
- the materials are not stressed beyond their elastic ranges.

In the list of predefined landing gear geometries there is also the Boeing 737-800, which is the critical airplane for Palermo International Airport, according to the Report dated back to 2009 (Laboratorio Centrale Ponti e Strade, 2009), being the one which requires the greatest pavement thickness.

The leaf output is a text document containing four parts:

- pavement layered structure table;
- aircraft gear data table;
- tire data table with loads;
- calculation results by evaluation points in terms of stress, strain and displacement.

The modulus values results are reported in Table 4.14.

Station/Drop	Layer	Alignment 1	Alignment 2
Station 1, Drop 3	E1	829.8	731.7
	E2	5484.9	2540.5
	E3	3968.1	1739.2
	E4	2866.0	2347.0
Station 2, Drop 6	E1	841.2	1152.2
	E2	10664.8	11240.6
	E3	6098.8	446.0
	E4	2021.7	1268.9
Station 3, Drop 9	E1	1424.3	1129.8
	E2	8684.2	10133.5
	E3	2508.8	657.9
	E4	644.4	700.3
Station 4, Drop 12	E1	1344.1	1380.3
	E2	5168.7	69421.2
	E3	3104.3	141.1
	E4	616.4	8051.5
Average Values	E1	1199.4	1004.5
	E2	6445.9	7971.5
	E3	3193.7	947.7
	E4	1375.6	1438.7

Tab. 4.14 Modulus values

The asphalt temperature on the day of the HWD tests was 30°C. For this temperature it is possible to say that the modulus values for asphalt mixture courses (represented by E1 in Tab. 4.14) are in line with the values obtained with laboratory tests, that is the average modulus values obtained by back-calculation are within the range of modulus values obtained with modulus tests and represented by the master curves in Figures 4.7-4.9.

4.4 Conclusions

This study has shown an application of mixtures with polymeric additives in the surface, binder and base courses of the access pavement to the new apron of Palermo International Airport, not yet open to air traffic.

Both field test and laboratory test results have confirmed the high modulus values obtained using additives in the asphalt mixtures and thus justify their use in settings such as airports, where the pavements are subject to high loads, which leads to gradual surface degradation, as well as to geometric boundaries, due to the complex and rigorous international standards that rule the civil aviation and the airport area.

Conclusions

This experimental study has led to important conclusions regarding the use of polymeric additives in asphalt mixtures.

The research activity carried out at the Road Materials Laboratory of the University of Guimarães made it possible to compare two different processes (WET and DRY) used for production of modified asphalt mixtures with polymers. Based on the results it was chosen the dry process in order to continue the study regarding polymer-modified mixtures, by carrying out further research on various polymers, included the waste ones, at the Road Materials Laboratory of the University of Palermo. Finally the research activity was completed with a case study at real scale.

In particular, in the research activity carried out at the Road Materials Laboratory of the University of Guimarães the results obtained indicated that polymer-modified mixtures showed similar or improved performance when compared to that of a conventional control mixture produced with harder virgin grade bitumen, not always available, or available at higher costs, in several countries. Thus, modifying asphalt mixtures with these plastic wastes can be an economic and ecological alternative for paving works. Moreover, the mixtures produced via the dry process showed increased water sensitivity and stiffness modulus properties.

The mixtures optimized in the research activity carried out at the Road Materials Laboratory of the University of Palermo showed good stability values and compaction and the positive influence that the additives studied have on these mixtures regarding permanent deformation resistance. Moreover, the presence of the additives allowed optimization of mixtures with lower binder contents and also made it possible to obtain higher stiffness modulus values than for traditional mixtures without additives as well as the values of the high modulus asphalt mixtures.

Finally, the results of the last case studied confirmed the results of the previous cases and thus justify their use in settings such as airports, where the pavements are subject to high loads, which leads to gradual surface degradation, as well as to geometric boundaries, due to the complex and rigorous international standards that rule the civil aviation and the airport area.

This study doubtless presents a series of innovative approaches and results when studying asphalt mixture modification. In particular, waste polymers were successfully used instead of virgin polymers; the DRY process of incorporation showed interesting performance, in comparison to the traditional WET process, mainly due to the use of a soft-base bitumen; the importance of selecting appropriate polymers was highlighted.

The WET process requires modification of the binder with specific equipment for blending bitumen and polymers at high temperatures, while no modification is needed for the DRY process. Therefore, the DRY process can become a good alternative to the usual binder polymer modification, since it can be easier and cheaper to implement, and results in mixtures with similar performance to those produced with the WET process. This could be particularly important for developing countries, since it widens the possibility of using locally available bitumens (of variable quality) and asphalt plants for producing mixtures with higher performance.

A possible future development is a thorough statistical study considering several kinds of bitumen and locally available aggregate sources, as locally available in order to highlight performance sensitivity to aggregate-bitumen combinations, as well as considering many other types of polymer, particularly wastes, since, as mentioned above, they can definitely be an economic and ecological alternative for paving works.

Appendix

Photographs

This appendix collects all images regarding the testing equipment, the materials and all that was used in the experimental phase of this dissertation.

Fig. A.1 Sieving machine (Guimarães)

Fig. A.2 Aggregate fraction size used (Guimarães)

Fig. A.3 Filler (Guimarães)

Fig. A.4 Tests on bitumen: A) Penetration Test, B) Ring and Ball Test, C) Resilience Test, D) DSR Test (Guimarães)

Fig. A.5 Automatic impact Marshall compactor (Guimarães)

Fig. A.6 Marshall specimen height measuring device (Guimarães)

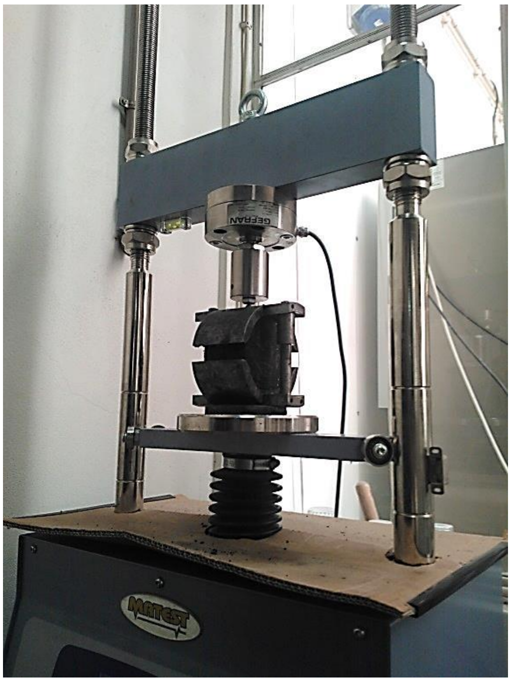

Fig. A.7 The Marshall Stability test (Guimarães)

Fig. A.8 The ITS test on specimens subjected to the water sensitivity test (Guimarães)

Fig. A.9 The stages of compaction by roller compactor (Guimarães)

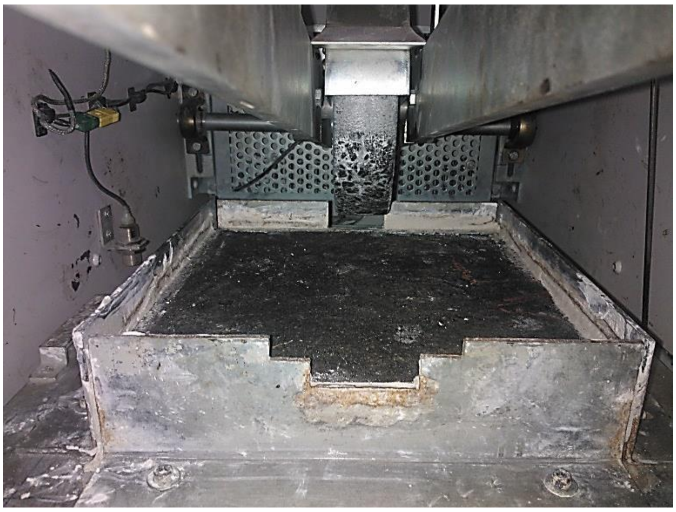

Fig. A.10 The wheel tracker test (Guimarães)

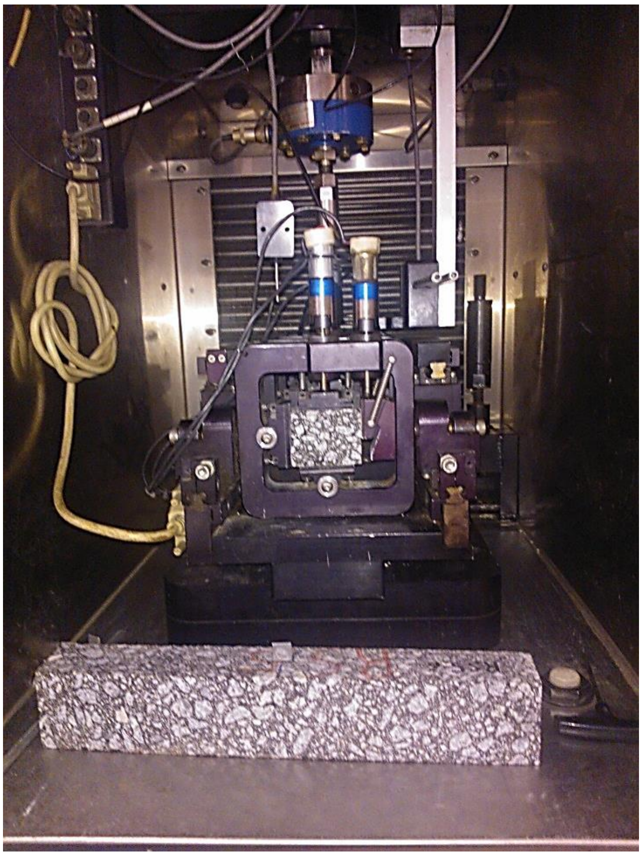

Fig. A.11 The four-point bending test device (Guimarães)

Fig. A.12 Automatic impact Marshall compactor (Palermo)

Fig. A.13 Thermostatic water bath before the Marshall Stability test (Palermo)

Fig. A.14 The Marshall Stability test (Palermo)

Fig. A.15 Hydrostatic balance (Palermo)

Fig. A.16 Gyratory compactor (Palermo)

Fig. A.17 The compaction of slabs for WTT by roller compactor (Palermo)

Fig. A.18 The stages of a wheel tracker test (Palermo)

Fig. A.19 The roller compactor for slabs (Palermo)

Fig. A.20 Slab after compaction (Palermo)

Fig. A.21 Specimen subjected to the four-point bending test (Palermo)

Fig. A.22 The four-point bending test device (Palermo)

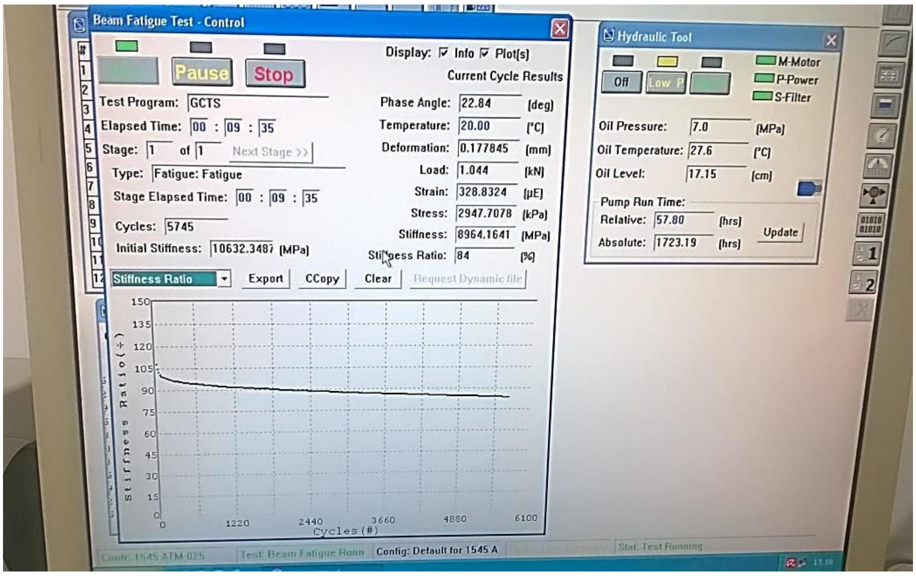

Fig. A.23 Screen CATS software during a 4PB test (Palermo)

Fig. A.24 Specimen subjected to the modulus test (Palermo)

Fig. A.25 Modulus test by means of a triaxial cell (Palermo)

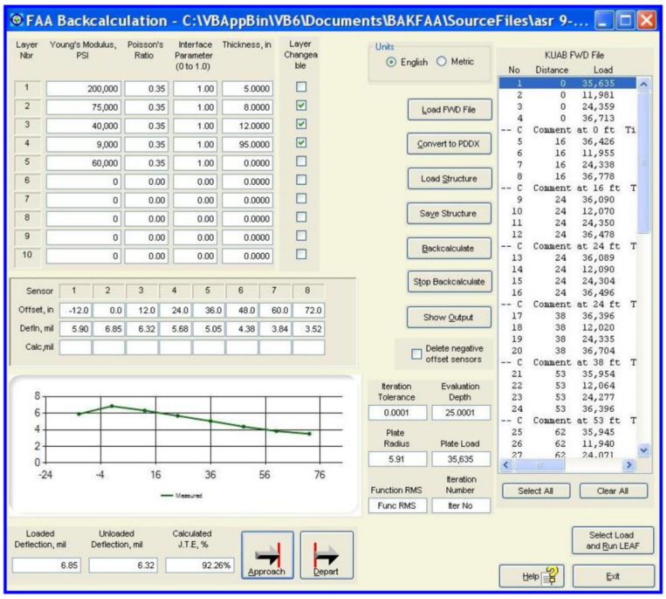

Fig. A.26 Screen BAKFAA software (Palermo)

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