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Low-Temperature Performance of Polymer-Modified Binders in Stone Mastic Asphalts

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Abstract: When temperatures drop to significantly low levels, road pavements are subjected to thermally-induced stresses, resulting in the appearance of thermal cracking, among other distresses. In these situations, polymers can be used as asphalt binder modifiers to improve certain asphalt binder properties, such as elastic recovery, cohesion, and ductility. Polymers also minimize some of the problems of asphalt mixtures, such as thermal and fatigue cracking and permanent deformation. This work's objective was to study the behavior of asphalt mixtures at low temperatures, mainly when using modified binders. Thus, three binders were selected and tested: a standard 50/70 penetration grade bitumen and two polymer-modified binders (PMB), obtained by adding, respectively, 2.5% and 5.0% of styrene–butadiene–styrene (SBS) in the 50/70 pen grade bitumen. Then, the PMBs were incorporated into stone mastic asphalt mixtures (namely SMA 11), which were subjected to low-temperature mechanical tests based on the most recent European Standards. The asphalt binders and mixtures evaluated in this work were tested for thermal cracking resistance, creep, elastic recovery, cohesive strength, and ductility strength. Overall, it is concluded that the studied asphalt mixtures with PMB, with just 2.5% SBS, performed adequately at low temperatures down to -20°C .

Keywords: low-temperature performance; polymer-modified binder (PMB); styrene–butadiene–styrene (SBS); stone mastic asphalt (SMA)



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1. Introduction

This paper aims to evaluate the influence of asphalt binder and its modification in the asphalt mixtures' low-temperature performance. Several laboratory methods and techniques were used to characterize the asphalt binders and obtain the tensile strength properties of asphalt mixtures at cold temperatures.

Stone mastic asphalt (SMA) was selected for this study as an excellent example of the current type of asphalt mixtures used in cold climate regions. Moreover, styrene–butadiene–styrene (SBS) modification of asphalt binder and its effect on the low-temperature performance of SMA were evaluated.

This paper should contribute to depict as closely as possible the actual conditions and behavior to which asphalt pavements are subjected when low temperatures are present, thereby expanding the current knowledge about the variables involved in the process. Subsequently, this work aims to add some knowledge on the response to challenges transport infrastructures face in regions with cold climates.

1.1. Low-Temperature Performance of Asphalt Mixtures

During freezing winters, road pavements are subjected to thermal stresses that can cause cracking of asphalt layers. Cracking can also occur due to the thermal fatigue of road surfaces in the alternating cycles of heating and cooling. Cracking by extremely low temperatures is a primary problem in icy regions [1]. Cracking due to thermal susceptibility

at low temperatures can also result from asphalt binders or mixtures retraction, which occurs when their maximum tensile strength is reached. Open cracks allow water to penetrate, which gradually freezes and expands and causes a partial destruction of the asphalt layers due to this volumetric expansion [2].

Polymers can be used as additives in asphalt mixtures so that their characteristics can be improved to achieve better performance [3,4]. Some properties of asphalt binder targeted in this work, such as elastic recovery, cohesion, and ductility, are potentially improved using polymer-modified binders (PMBs). PMBs can also minimize some of the main problems of asphalt mixtures, such as permanent deformation and cracking due to thermal susceptibility and bitumen aging [5,6].

The stresses in the asphalt layers derived from the low temperature during cooling are fairly challenging to measure directly in the pavement structure. Therefore, the correct analytical estimate and the results obtained through laboratory tests are crucial since they can be compared to the empirical results to calibrate and rectify both [7].

The pavements' surface temperature can vary significantly with weather factors such as radiation, wind speed, atmospheric pressure, and low ambient temperature. In contrast, the asphalt mixture's heat transfer properties have little effect. Wang et al. [8] stated the importance of determining the external factors accurately in order to simulate the pavement surface temperature variation during testing conditions. More substantial temperature changes are recorded closer to the pavement surface, leading to higher thermal stresses. Even though the magnitude of the temperature variation in winter is smaller than in summer, the pavement is subjected to freezing and thawing cycles. The most critical condition in the surface layer, in winter conditions, is the diurnal temperature variation [8].

Low-temperature cracking is one of the most common failure modes of flexible pavements in cold climate zones [1,9] and should be thoroughly studied, thus driving some recent works' attention [10,11]. These regions are characterized by daily temperature drops that can be remarkably rapid, very long cold seasons, and the incidence of lowest temperatures. All these factors will justify the continuous maintenance needed to keep the roads in acceptable service conditions, resulting in high direct and indirect costs.

In colder regions, pavements must endure not only when it comes to the low levels of temperature, considering the brittle nature of bitumen in that circumstance, but also to the cooling pace. The cooling rate can create tensile stresses derived from the contraction of material when cooled. These stresses occur because the asphalt layer is constrained in the pavement structure and cannot relieve the thermal stresses by internal relaxation. Due to the drop in temperature, thermal tensile stresses increase, and microcracks due to low temperature may appear on the pavement surface after exceeding the mixture's fracture or tensile strength [12]. If the low-temperature cycles continue, which is the usual scenario in colder regions, these microcracks propagate through the pavement, leading to a more severe cracking problem in the mixture [13].

In colder zones, when water fills these cracks during the winter months, it freezes, and ice lenses along with frost heave can develop. This condition results in the loss of fines and the formation of voids throughout the pavement, leading to a load-bearing capacity reduction. If these are not adequately considered, they can lead to the pavement having a reduced service life, high maintenance costs, and poor riding quality [14]. This condition and the change in the microstructural stress mechanism determine the fracture resistance behavior of asphalt mixtures to low temperature [15]. However, other pavement distresses, such as rutting, moisture-induced damage, among others, may appear on asphalt pavements that have been in use for an extended period [1,16,17].

Several laboratory tests were developed to assess the factors that affect asphalt mixtures in colder regions. One of these methods is the thermal stress restrained specimen test (TSRST), which determines the critical cracking temperature that results from a single drop to a very low-temperature value observed during severe winters [18]. The test presents many advantages and some limitations, such as the fact that failure temperatures obtained from the test depend on the cooling rate [12].

Other characteristics can be tested by different methods, such as the direct tensile strength of asphalt mixtures at low temperatures measured with the uniaxial tension stress test (UTST) or the creep compliance assessed with the indirect tensile test (IDT) [12]. The bending beam rheometer (BBR) can also obtain the asphalt binder strength at low temperatures [19]. Another method to describe the low-temperature strength properties of asphalt mixtures is the bending beam test (flexural strength) at low temperatures [20].

Fracture properties of asphalt pavements can be defined based on fracture mechanics theory and strictly related to laboratory test results. There are several test methods to assess fracture parameters, including the bending of single edge and notched beams (SENB), bending of semi-circular beams (SCB), and tension of disc-shaped compact tension (DCT) specimens. However, SCB is the most suitable and frequently used method. Several parameters can be assessed for better cracking characterization, such as fracture energy (pre- and post-peak), toughness index, and flexibility index [12].

Another indicator of crack formation's potential is the relaxation potential of a mixture, i.e., the capacity to dissipate the thermal-induced stress. This capacity plays a vital role since the higher the relaxation potential, the lower the risk to reach the failure point and form cracks. Mixtures characterized by faster decay of relaxation modulus are less prone to cracking at low temperatures and more suitable for cold climate regions [9].

1.2. Characteristics of Stone Mastic Asphalt

SMA presents several benefits because it can reduce light reflection, improve surface drainage and reduce the level of traffic noise. The discontinuous skeleton can offer an improved dispersion of the traffic loads, prevents rutting, and increases wearing resistance. Likewise, it also provides good macrotexture and high surface roughness, contributing to improving skid resistance [21]. Research over the years found that stripping, surface cracking (either induced by temperature fluctuation or by traffic loads), and raveling do not present severe issues in this type of mixture [22]. SMA mixtures' excellent durability derives from the mastic mortar's impervious nature, which is very rich in asphalt binder, in which the type of filler plays a relevant role [23]. The improved performance of SMA mixtures is also justified by the general use of stabilizing agents such as fibers or bitumen modifiers [24,25].

However, SMA mixtures retain some drawbacks, including the potential for fat spots and bleeding, which can be solved using fibers. Another consideration is its higher costs due to the materials used in SMA mixtures, particularly the high amount of binder and use of additives [26]. Despite the initial cost, the return can be positive since the initial investment can be recovered in savings on repairs and maintenance due to its higher durability. In its initial phase, another weakness of SMA is its lower sliding resistance derived from production or application errors [21].

The most used SMA types in European countries are SMA 8, SMA 11, and SMA 16, and such countries include Germany, the Netherlands, and Sweden. Nordic countries use this type of mixture to increase the resistance to studded tires [24]. SMA 11 is significantly used by nations in colder regions, justifying its use in this work. According to Judycki [27], SMA is becoming the most popular surface course on newly constructed main roads in Europe.

According to European Asphalt Pavement Association [24], the binders utilized in SMA are usually polymer-modified bitumens and standard bitumens with fibers, with the latter being used on lower traffic volume roads.

1.3. Polymer Modification of Asphalt Binders

Polymers are widely used as modifiers in asphalt polymer-modified binders (PMBs). They include a broad range of subclasses that can meet the demands imposed by asphalt mixtures requirements, including elastomers and plastomers [28]. Recycled crumb rubber can also improve asphalt binder performance in conventional or recycled mixtures [29].

Styrene-butadiene-styrene (SBS) is a block copolymer of styrene and butadiene [30]. These types of polymers comprise alternating segments of different compositions, which

are linked together through their reactive ends. In this copolymer, while polystyrene (PS) blocks impart strength, soft polybutadiene (PB) blocks provide elasticity [31].

PB block structure can be adjusted using special catalysts that partially transfer the double bonds on the PB blocks to the side chains. According to Vonk et al. [32], this modification can provide many advantages for the PMB, such as low viscosity, better compatibility at equivalent molecular weight, resistance to oxidation, and thermal stability.

SBS can be considered a thermoplastic elastomer, and its glass transition point can characterize the main change in its thermal properties as an amorphous polymer. One of the primary advantages of elastomer modification is that the effectiveness of the cross-links reduces rapidly above the PS glass transition point at about 100 °C. The PS domain reforms on cooling, and the strength and elasticity are restored [33]. SBS absorbs the maltenes in bitumen, swells, and forms a continuous molecular network at higher dosages, which makes up a significant fraction of the bitumen. These networks increase the binder’s elastic component, which provides a better recovery after deformation [34].

In sum, the advantages of SBS copolymer in bitumen modification are essential in the current work due to its higher flexibility at low temperatures, which inhibits cracking and improves the binder’s resistance to crack reflection. In addition, SBS modification ensures better flow and rutting resistance at high temperatures, improved strength, and excellent elasticity [35]. However, it presents some challenges, such as higher cost, higher viscosity at compaction temperatures, lower resistance to heat, and some oxidation problems [36].

2. Materials and Methods

2.1. Materials

Stone mastic asphalt with a maximum aggregate size of around 11 mm (referred to as SMA 11) was selected for evaluating the mechanical performance at low temperatures.

The aggregates and the other components were used with the same mix design for all the studied asphalt mixtures. The properties of these aggregates follow the specifications provided by the European Standard EN 13043. The local availability of the aggregates used in this work was considered. Natural igneous rocks are abundant in the region, and thus the aggregates selected are mainly of granite origin, except the filler, which is limestone, selected to improve the mixture workability. The particle size distribution was determined by sieving the material according to the procedure specified in EN 933-1.

The following task consisted of obtaining the final grading composition required for the selected asphalt mixture, SMA 11, by determining the amount of each aggregate fraction that should be incorporated in the mixture. This task was executed by taking into account a standard minimum and maximum proportion for each particle size for this specific type of asphalt mixture, as shown in Table 1.

Table 1. Grading envelope of the selected asphalt mixture.

Sieve Size (mm)	SMA 11 (%)
16	100
11.2 (11)	90–100
8	55–80
4	22–33
2	20–30
0.5	12–20
0.063	6–10

A polymer commonly used for binder modification (styrene–butadiene–styrene, SBS) was used for both asphalt mixtures in different proportions of the mixture’s total weight. This polymer is white, and its grain size varies from approximately 3 to 6 mm, with most of the particles having dimensions around 3 mm. The SBS information declared by the supplier grants it a styrene content of 30%, a hardness (Shore A) higher than 77, elongation

at break higher than 760%, and a tensile strength higher than 26 MPa. For the production of both polymer-modified binders, the same type of virgin bitumen was used as the base binder, which was a 50/70 penetration grade bitumen.

Two asphalt mixtures were produced with the same constituents to compare the SBS polymer's influence in the asphalt mixtures' behavior; only the polymer amount was varied. This variation entails two proportions of SBS polymer incorporation in the asphalt binder: (i) the first one, hereafter named PMB25, comprises 2.5% SBS; (ii) the second one, hereafter named PMB50, comprises 5.0% SBS.

2.2. Production of Modified Binders

The wet method was selected as the asphalt binder modifying process. For the modified binders' production, the base bitumen was first heated up to 170 °C, and the polymer was incorporated using a low-shear mixer, i.e., the IKA RW 20 DZM, so that the polymer could be added slowly and more homogeneously. The container where the modified bitumen was being blended was then transferred to a high-shear mixer, i.e., the IKA T5 Ultra Turrax Basic, where the actual modification process could be performed. The bitumen and polymer blend was continuously mixed at a constant rotation of 7400 rpm, with a constant temperature of 170 °C maintained for approximately 90 min.

These conditions allow the polymer to be adequately digested in the bitumen to obtain the final polymer-modified binder. The high shear mixer can thoroughly crush the SBS polymer granules because of its rotor-stator geometric properties.

2.3. Characterization of the Asphalt Binders

2.3.1. Softening Point and Penetration

The tests for characterization of a polymer-modified bitumen used in SMA mixtures must comply with the European Standard EN 14023, which includes standards for penetration and softening tests, among others. This PMB standard combines classes that specify PMBs' basic requirements that can be used in SMA mixtures [21].

The ring and ball method is utilized to evaluate the softening point of an asphalt binder, according to EN 1427. The needle penetration determination method is a test standardized by EN 1426. This procedure indirectly determines the consistency—or more specifically, the relative hardness—of the bitumen and asphalt binders.

2.3.2. Rheological Characterization

The asphalt binders' rheological properties at high and intermediate temperatures were determined using the dynamic shear rheology test. This test was based on EN 14770, and the equipment used for this procedure was a dynamic shear rheometer (DSR) by Bohlin Instruments. The process involves determining the complex shear modulus, G^* , and the phase angle, δ , of the asphalt binders over a range of test frequencies and temperatures when tested in an oscillatory shear. The DSR applies controlled oscillatory strains to bitumen samples at a frequency of 10 rad/s while measuring the torque applied to achieve these strains. Strain levels of 0.1% (46 to 88 °C) and 0.05% (19 to 40 °C) were selected to ensure specimens testing in the linear viscoelastic region over the temperature range and frequency chosen. Knowing the torque effort and the respective torsion deformation makes it possible to determine the bitumen's rheological properties.

The binders were tested at different temperatures, ranging from 19 to 88 °C, with the plates changed according to the tested temperatures. For temperatures between 19 and 40 °C, an 8 mm diameter plate was used with a bitumen sample with 2 mm of thickness. For temperatures between 46 and 88 °C, a 25 mm diameter plate was used with a bitumen sample with a thickness of 1 mm. This change of the plates ensures a linear rheological behavior of the bitumen.

2.3.3. Elastic Recovery

The elastic recovery measures an asphalt binder specimen’s capacity to return to its original position after being stretched to a specified elongation by applying tensile stress. The method that allows this measurement to be taken is also named the elastic recovery method and is essentially a procedure performed to determine the elastic recovery of asphalt binders in a ductilometer at a specific test temperature. It is especially applicable to PMBs modified with thermoplastic elastomers but can also be used with other asphalt binders, even though these will generate a minor recovery. This process is carried out according to EN 13398. Samples must be stretched at a speed of 50 mm/min at the stipulated temperature until they reach a predetermined elongation of 200 mm to determine the elastic recovery. Then, they are cut in the central section and, after 30 min, the distance recovered between the two halves is measured (Figure 1).



Figure 1. Elastic recovery test of polymer-modified binders: molds and measuring procedure.

This test should be carried out in a temperature-controlled water bath that is capable of maintaining the specimen and the attachment device at the specified temperature throughout the test to an accuracy of ± 0.5 °C. The test must be carried out on at least two specimens in parallel, which should be conditioned in the water bath at the test temperature for at least 90 min. In the current work, the test samples were prepared following the European Standard EN 12594, as specified in EN 13398. The test was performed at two test temperatures (5 and 20 °C) for two polymer-modified binders (PMB25 and PMB50).

2.3.4. Force Ductility

Ductility is a material’s ability to undergo a visible enduring deformation through elongation (decreasing the cross-sectional area) without breaking. It expresses the extent to which the material can be plastically deformed without fracture. In practical terms, it is defined as the distance in millimeters to which a standard sample of the material can be stretched without breaking. The method was performed according to EN 13589. The asphalt binder sample must be stretched at 50 mm/min in a ductilometer at the test temperature until it reaches a predetermined elongation of 400 mm or until it breaks (Figure 2).



Figure 2. Force ductility test of PMBs: molds and specimens elongated.

The testing conditions are similar to those of the elastic recovery test. Cohesion is a measure of the tensile stress required to break the bond between the asphalt binder molecules. The inherent strength, tenacity, and toughness of the asphalt binders can be

improved by modification with thermoplastic polymers or rubber crumbs. A higher tensile stress is required to break the molecular bonds of modified binders and cause failure, compared with lower tensile stress required to break conventional binders' bonds [34].

This method was performed at two test temperatures (5 and 20 °C) for both polymer-modified binders studied in this work (PMB25 and PMB50).

2.4. Production and Characterization of Asphalt Mixtures and Specimens

2.4.1. Asphalt Mixture Production

The asphalt mixtures were selected and produced after performing several laboratory tests to characterize both the aggregates and the asphalt binders, following the specifications provided by the European Standards EN 13108-1 and EN 13108-5. This process is divided into two parts: the first phase for mixture production and the second phase for slab compaction. These asphalt mixtures for surface courses were prepared with the two PMBs studied in this work, using the same aggregates.

2.4.2. Specimens Production

The specimens' dimensions that were used to evaluate their low-temperature mechanical characteristics are described in the European Standard EN 12697-46. The specimen must be either a prismatic or a cylindrical column. In the present work, a prismatic specimen with a 160 mm height and a square cross section of 40 mm was selected, given that the maximum aggregate size of the studied mixtures is 11.2 mm.

The specimens were obtained by sawing asphalt slabs, which were precompacted using a roller compactor according to EN 12697-33. The specimens were mainly sawed from the slab central part to ensure the required distance of 20 mm to its border.

2.4.3. Specimen Characterization

Test specimens can be characterized by calculating the air voids content in the compacted asphalt mixture and other volumetric-related properties. Some of those properties are the maximum density, together with the bulk density and, of course, the air void content. These properties were obtained according to European Standards EN 12697-5, EN 12697-6, and EN 12697-8.

2.5. Low-Temperature Mechanical Characterization of the Asphalt Mixtures

The tests carried out to characterize the low-temperature performance were based on European Standard EN 12697-46. A brief description of the tests is made in the following sections, which focus on the specific conditions selected for this work (e.g., testing temperatures).

2.5.1. Testing Conditions and Apparatus

The mechanical methods used to characterize the specimens require an apparatus consisting of a thermostatic chamber with forced air circulation where the specimen can be conditioned and tested. The apparatus used was a servo-controlled universal testing machine (CS7400S) with fixed endplates. It has a load cell connected to the vertical actuator with a capacity of 22 kN, and it is suitable for specimens with a height up to 160 mm (Figure 3). Its temperature chamber can operate between -20 and 30 °C (± 0.5 °C) and can change the specimen temperature at a rate of 10 °C/h. The equipment's limitation to adequately operate below -20 °C defined the lowest testing temperature used in this work.

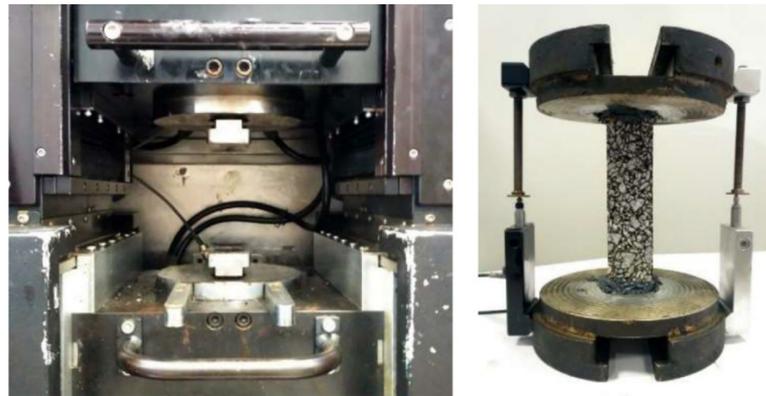


Figure 3. Testing apparatus and specimen ready for testing.

2.5.2. Uniaxial Tension Stress Test

In the uniaxial tension stress test (UTST), a specimen is pulled with a constant strain rate of 1 mm/min at a constant temperature (T) until failure. Four different test temperatures were selected to perform these tests, namely -20 , -10 , 5 , and 20 °C. Results of the UTST are the maximum tensile strength $\beta_t(T)$ and the corresponding tensile failure strain $\varepsilon_{\text{failure}}(T)$ at the test temperature T . For each temperature, three specimens were tested in order to have a more precise range of results.

2.5.3. Thermal Stress Restrained Specimen Test

In the thermal stress restrained specimen test (TSRST), the specimen, whose length is held constant, is subjected to a decrease in temperature with a constant temperature rate. Cryogenic stress is built up in the specimen due to the prohibited thermal shrinkage. This procedure allows us to obtain results concerning the progression of the cryogenic stress over the temperature $\sigma_{\text{cry}}(T)$ and the failure stress σ_{cry} , at the failure temperature T_{failure} . In this test, the failure stress is equivalent to the specimen's strength at the failure temperature [12].

This test was performed with three different temperature decrease rates to observe the effect this cooling rate has on TSRST results and, indirectly, on the tensile strength reserve analysis, explained further ahead. The starting temperature, $T_0 = 20$ °C, as recommended for initiating the test, was observed, and the cooling rate indicated in the standard was used, i.e., $\Delta T = -10$ °C/h. According to Pszczola et al. [37], the cooling rate significantly affects this test's experimental measurements. To determine this effect, two additional cooling rates were selected: $\Delta T = -2$ °C/h and $\Delta T = -5$ °C/h. This selection was made based on the research made by Pszczola et al. [38], in which the pavement temperatures were analyzed in a country with a climate representative of low-temperature conditions. In that study, it was observed that a cooling rate of -2 °C/h has a probability of occurrence of 99%, which can be a good representation of the typical low-temperature conditions during a considerable portion of the winter.

Based on UTST and TSRST tests and their results, it is possible to determine the tension strength reserve, $\Delta\beta_t(T)$, which the mixture can support under external loads in addition to the low-temperature thermal stresses. It is calculated as the difference between the tensile strength, $\beta_t(T)$, which was obtained from the UTST as a temperature/tensile strength diagram using a cubic spline function, and the cryogenic stress $\sigma_{\text{cry}}(T)$, which was obtained from the TSRST at the same temperature T using Equation (1).

$$\Delta\beta_t(T) = \beta_t(T) - \sigma_{\text{cry}}(T), \quad (1)$$

where $\Delta\beta_t(T)$ is the tensile strength reserve (in MPa), $\beta_t(T)$ is the tensile strength (in MPa), and $\sigma_{\text{cry}}(T)$ is the cryogenic (thermal) stress (in MPa).

2.5.4. Relaxation Test

In the relaxation test (RT), the specimen is subjected to an instantaneous strain ϵ held on a constant level. The decrease of tension stress by relaxation over the testing time is monitored. The initial stress should not be higher than 75% of the tensile strength β_t obtained from the UTST tests. The relaxation time corresponds to the time measured between the moment the strain is imposed and the stress reduction to $36.8 \pm 0.1\%$ of its initial value ($1/e = \frac{1}{2} \cdot 718 = 0.368$). These test results are the relaxation time t_{rel} and the remaining tensile stress σ_{rem} after the test.

2.5.5. Tensile Creep Test

In the tensile creep test (TCT), the specimen is exposed to a constant tension stress σ at a constant temperature T , during which the progression of the strain ϵ is measured. After a given time, the stress is withdrawn, and the strain regression continues to be measured. Rheological parameters describing the asphalt’s viscoelastic properties can be determined by interpreting the strain measurements. It is recommended to hold the constant load for eight hours and record the regression after unloading for an additional two hours.

Table 2, presented in EN 12697-46, suggests the test temperatures and suitable stress levels as a proportion of the tensile strength obtained in UTST. Moreover, since one of the selected test temperatures is $-20\text{ }^\circ\text{C}$, the corresponding stress level proportion was interpolated between the adjacent values.

Table 2. Recommended test conditions for TCT.

Temperature ($^\circ\text{C}$)	Proportion of $\beta_t(T)$ (%)
-25	50
-20	43
-10	30
+5	10
+20	5

2.5.6. Uniaxial Cyclic Tension Stress Test

In the uniaxial cyclic tension stress test (UCTST), a specimen is subjected to cyclic tensile stress, characterized by sinusoidal stress, to simulate the dynamic loading condition by traffic in combination with constant cryogenic stress. During the test, the strain response is monitored, and the evolution of stiffness is recorded until fatigue failure occurs. The test results are the number of applied load cycles until failure ($N_{failure}$) and the number of load cycles until the conventional fatigue criterion $N_{f/50}$ is reached.

The fatigue behavior can be characterized by relating the strain of a mixture to the number of load applications to failure. Fatigue testing can be conducted by either controlling the load (stress) or the deformation (strain) [39].

The UCTST must be carried out at one or more test temperatures T and testing frequencies (f). It is recommended to use a testing frequency of 15 Hz and a test temperature of $-15\text{ }^\circ\text{C}$. However, the testing machine CS7400S is only being able to maintain a stable frequency of 10 Hz. Thus, all the tests were performed at that frequency. Concerning the temperatures and maintaining the same logic, they were the same as all other mechanical tests. The base stress at test temperature T should be derived from the TSRST, where the base stress corresponds to the cryogenic tensile stress $\sigma_{cry}(T)$ at test temperature T . The maximum stress σ_{tot} should be the sum of the base stress $\sigma_{cry}(T)$ and the stress caused by the traffic load $\Delta\sigma$. Surface course asphalt mixes should use traffic stress of $\Delta\sigma = 1.6\text{ MPa}$.

3. Results and Discussion

3.1. Aggregate Particle Size Distribution of the Studied Mixture

The optimal grading curve for an SMA 11 mixture was obtained based on all available aggregates’ particle size distribution. All mixtures were produced with identical amounts

of each aggregate to limit the variables affecting the results. The selected optimal grading composition percentages of each aggregate type are:

- Limestone filler = 8%;
- Granite aggregate 0/4 = 20%;
- Granite aggregate 4/6 = 15%;
- Granite aggregate 4/10 = 42%;
- Granite aggregate 6/14 = 15%.

The optimized grading composition curve of SMA 11 mixtures used in this work is shown in Figure 4. As can be observed, the most used aggregate type is the 4/10, with 42% of the total. This situation is typical for SMA 11 mixtures, which will somewhat reflect the discontinuous gradation of the aggregates in this mixture that presents a stone-to-stone skeleton held together by a rich mix of binder and filler [40]. The high amount of filler (8%) used in this mixture is essential for SMA mixtures’ mastic component.

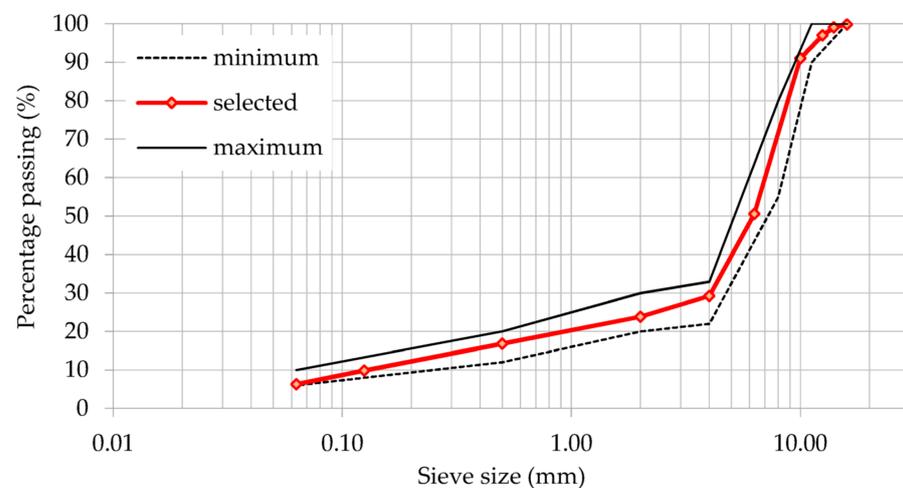


Figure 4. Selected grading composition curve for stone mastic asphalt with a maximum aggregate size of around 11 mm (i.e., SMA 11) mixtures.

3.2. Analysis of the Asphalt Binders

3.2.1. Softening Point and Penetration

The initial tests performed to characterize the asphalt binders’ physical properties were the softening point, the ring and ball method, and the penetration test using the needle penetration method. The tests were performed on both PMBs with 2.5% and 5.0% SBS and the base 50/70 bitumen. Table 3 presents the results obtained from these tests.

Table 3. Penetration and softening point properties of the studied asphalt binders.

Asphalt Binder	Penetration at 25 °C (0.1 mm)	Softening Point (°C)
Base 50/70 bitumen	55.2	50.5
PMB25	42.7	90.9
PMB50	42.3	89.5

The results of the base bitumen 50/70 meet the standard limits of EN 12591. The softening temperature should vary between 46 to 54 °C, and the penetration must be between 50 and 70 tenths of a millimeter. Both results comply with the pre-established parameters.

Regarding the PMBs, both have a lower penetration value than the base bitumen and present similar penetration values (around 42 tenths of a millimeter). Thus, the PMBs are more viscous than the conventional base bitumen. Moreover, the PMBs have very

similar softening point temperature values (around 90 °C) but much higher than traditional base bitumen, as expected. This result indicates that these PMBs are less susceptible to temperature changes, which is a required attribute for this work’s purpose. Both PMBs are also substantially more stable to permanent deformation [41].

According to EN 14023, both PMBs evaluated in this work can be included in class 3 of penetration at 25 °C ($25-55 \times 0.1$ mm) and classified as class 2 (≥ 80 °C) with regard to the softening point temperature. They present similar performance and could be classified as PMB 25/55-80 hereafter. Nevertheless, the PMB25 and PMB50 terms will be upheld to maintain the same classification and to avoid confusion.

3.2.2. Rheological Characterization

The rheological analysis evaluates the phase angle, δ (°), and complex shear modulus, G^* (kPa), for a temperature range between 19 and 88 °C, using the dynamic shear rheometer (DSR) procedure according to EN 14770 standard. Figure 5 presents the complex shear modulus and phase angle of the asphalt binders used in this study: (i) base bitumen 50/70; (ii) the PMB with 2.5% SBS, PMB25; (iii) the PMB with 5.0% SBS, PMB50.

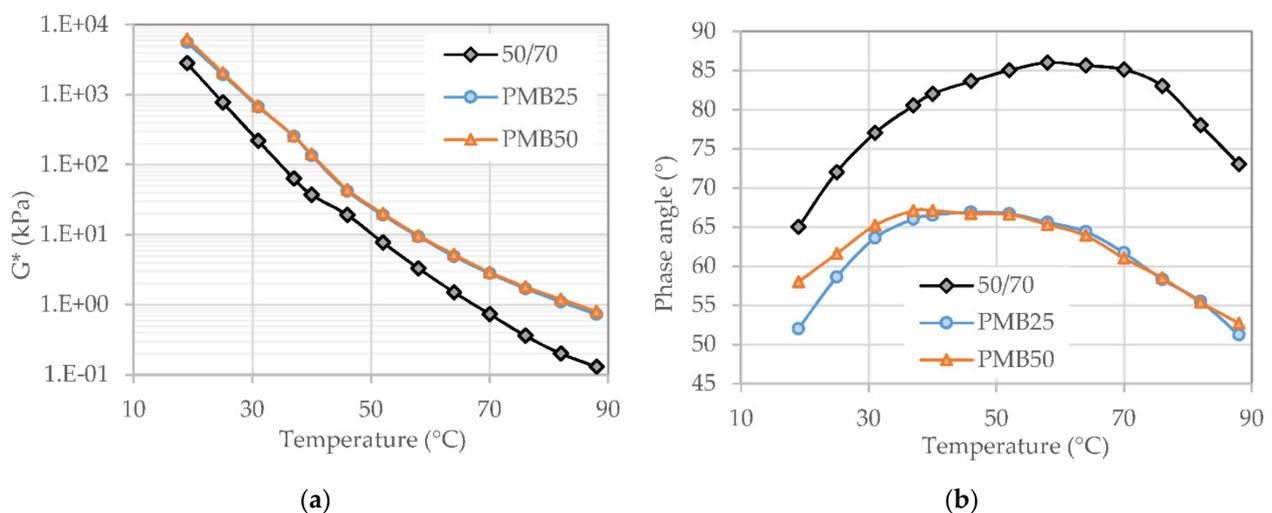


Figure 5. Rheological characteristics of the asphalt binders, i.e., the base bitumen 50/70, PMB with 2.5% SBS (PMB25), and PMB with 5.0% SBS (PMB50): (a) Complex shear modulus (G^*); (b) phase angle (δ).

The complex shear modulus results show that both polymer-modified binders have identical behavior concerning this parameter, as both curves superimpose almost perfectly. The PMBs present a higher stiffness than the base bitumen 50/70 over the temperature range analyzed. The differences become more evident at high temperatures, stressing the excellent performance of PMBs in those conditions. Thus, the SBS polymer used to modify both binders significantly increased the base bitumen’s complex modulus.

The phase angle, δ (°), provides a relative indication of the viscous and elastic behaviors of the asphalt binder, and this angle varies between 0° (utterly elastic) and 90° (utterly viscous). The three bitumens present phase angle curves above 45° and can be presented as having a more viscous rather than elastic behavior.

The PMB25 has the lowest phase angle at 20 °C, albeit not considerably different from the other binders. Thus, this bitumen presents a slightly less viscous behavior at this temperature than the other binders. When the temperature increases, both PMBs tend to have similar phase angle values. An increase in the phase angle values of the PMBs can be seen until they reach a maximum value of 67° at 46 °C, which indicates an increase in the viscous behavior until that temperature. The phase angle curves of both PMBs superimpose after 46 °C, with the viscous nature of the PMBs decreasing as the temperature increases.

The base bitumen 50/70 presents a similar behavior to the PMBs, albeit with higher phase angles and a maximum value of 86° obtained at higher temperatures (60 °C).

The base bitumen and PMBs' similar shapes of the phase angle curves show the base bitumen's contribution to the PMBs behavior. The polymer's effect on the base bitumen reduces the phase angle, improving the elastic behavior of the PMB. The smooth or undulating change in the phase angle over a broad temperature range can be primarily attributed to the use of modifiers [31], which improves the rutting resistance of asphalt mixtures.

Other authors used rheological tests on a DSR [42–44] or a bending beam rheometer (BBR) [45–47] to show that the addition of SBS significantly reduces the low-temperature stiffness of asphalt binders and improves its fatigue performance. Still, the SBS polymer is swollen by saturates and aromatics and can make the base binder lack maltene fraction.

3.2.3. Elastic Recovery

The elastic recovery test was performed according to EN 13398, for PMB25 and PMB50, at two test temperatures, i.e., 5 and 20 °C. These temperatures were selected to match those of the low-temperature performance tests later performed on SMA mixtures. The results obtained for the final elastic recovery value R_E are shown in Table 4.

Table 4. Results of elastic recovery, R_E (%), of the studied PMBs.

Parameter	5 °C		20 °C	
	PMB25	PMB50	PMB25	PMB50
Mean value	65%	51%	98%	95%
Standard deviation	1%	1%	0%	0%
Coefficient of variation	2%	1%	0%	0%

The minimum criterion of a predetermined elongation of 200 mm was reached for both PMBs at 20 °C, but three samples broke prematurely at 5 °C. The procedure detailed in the standard for brittle failures was followed in those situations.

PMB25 and PMB50 behaved remarkably well at 20 °C, with almost 100% elastic recovery. Although PMB25 has a lower SBS content, it presented slightly better results than PMB50, with a mean value of 98% against 95%. Both binders had lower elastic recovery values at 5 °C, given their high stiffness and brittle behavior at lower temperatures, while the higher elastic recovery of PMB25 (65%) against PMB50 (51%) becomes evident.

Overall, both PMBs presented an excellent elastic recovery behavior, which may result in a potential good low-temperature performance [48] of SMA mixtures.

3.2.4. Force Ductility

The force ductility method was performed according to EN 13589, for PMB25 and PMB50, at two test temperatures, i.e., 5 and 20 °C. As previously mentioned, these temperatures were selected to match those of the low-temperature tests on SMA mixtures. Figure 6 shows the force ductility graphs obtained for the tested asphalt binders in this method.

The force ductility (FD) tests of both PMBs performed at 20 °C followed the standard because all samples were elongated up to 400 mm. The force required to elongate the PMB50 was almost 1.5 times higher than that required by PMB25, demonstrating the higher tensile stress limit and ductility energy retained by the PMB50 at this temperature. However, at 5 °C, both PMBs could not elongate the established 400 mm, as they presented a premature break after 350 mm of elongation. Comparing the PMBs, they presented an equivalent performance during the test at 5 °C, with very high force ductility values.

Regarding the temperature's influence on the ductility results, the energy required to stretch the samples at 5 °C is nearly fourteen times higher than that needed at 20 °C. The force ductility curves' shapes are consistent with the strength increase expected in a PMB after the initial peak, and both PMBs presented an excellent ductility.

Mollenhauer and Tušar [49] found that the increase in FD maximum force can indicate higher bitumen stiffness, which results in reduced low-temperature cracking resistance of

the asphalt mixture or higher failure temperatures. Thus, the higher FD force of PMB50 can result in a worse low-temperature performance than that of PMB25.

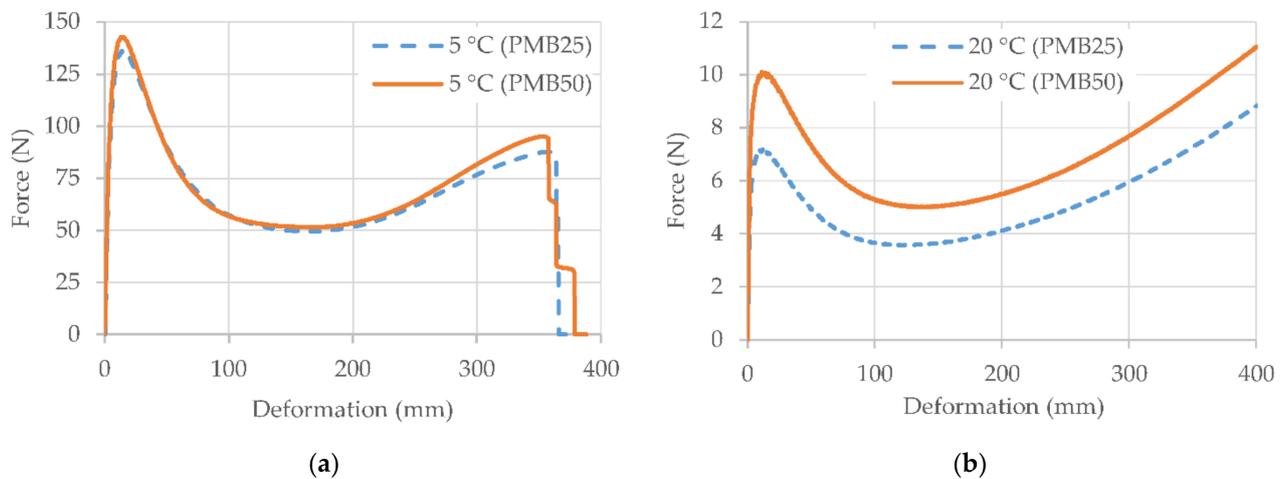


Figure 6. Force ductility results for the studied PMBs: (a) tests at 5 °C; (b) tests at 20 °C.

3.3. Analysis of the Asphalt Mixtures and Specimens

According to EN 13108-5, the designation for the two mixtures studied in this work is SMA 11 PMB25 (i.e., the mixture SMA 11 PMB 25/55-80 containing 2.5% PMB) and SMA 11 PMB50 (i.e., the mixture SMA 11 PMB 25/55-80 containing 5.0% PMB).

The air void contents of both asphalt mixtures were determined based on the bulk density of their compacted specimens. The maximum density was previously assessed for the loose PMB25 and PMB50 mixtures. These characteristics are summarized in Table 5.

Table 5. Volumetric characteristics of the studied asphalt mixtures.

Properties	Unit	SMA 11 PMB25			SMA 11 PMB50		
		Mean Value	Standard Deviation	Coefficient of Variation	Mean Value	Standard Deviation	Coefficient of Variation
Maximum density, ρ_{mv}	(Mg/m ³)	2.461	-	-	2.469	-	-
Bulk density, ρ_{bssd}	(Mg/m ³)	2.405	0.011	0.5%	2.399	0.015	0.6%
Air voids, V_a	(%)	2.27	0.46	20.1%	2.83	0.62	22.1%

The specification for SMA 11 mixtures requires air void contents between 2 and 5%, which was observed for both mixtures with PMB25 and PMB50. The air voids content has a significant influence on the strength and deformation properties of asphalt mixtures. Brown et al. [50] evaluated SMA mixtures applied in the United States from 1991 to 1993; they observed that the mixtures were typically designed to have approximately 3.5% air void content or less and appear to resist cracking better than other dense mixtures. They also concluded that generally, the air void content design is somewhat lower for the northern than for the southern states of the US, indicating that pavements subjected to lower temperatures for extended periods can benefit, to some extent, from lower air void content.

3.4. Mechanical Low-Temperature Performance of the Asphalt Mixtures

3.4.1. Uniaxial Tension Stress Test (UTST)

In this test, according to EN 12697-46, a specimen is pulled at a constant strain rate ($\Delta\epsilon$) of 6.25%/min or 1 mm/min for a constant temperature until failure (Figure 7). The results are the maximum stress before failure, or tensile strength, $\beta t(T)$, and the corresponding failure strain $\epsilon_{failure}(T)$ at each of four test temperatures used ($T = 20, 5, -10, \text{ and } -20\text{ }^\circ\text{C}$). In those conditions, the tensile strength results of the studied mixtures are shown in Table 6.



Figure 7. Example of tested specimen in the UTST test.

Table 6. Tensile strength results, β_t , from UTST for SMA 11 mixtures with PMB25 and PMB50.

Property	Test Temperature (°C)	SMA 11 PMB25			SMA 11 PMB50		
		Mean Value	Standard Deviation	Coefficient of Variation	Mean Value	Standard Deviation	Coefficient of Variation
Tensile strength β_t (MPa)	−20	5.437	0.466	9%	-	-	-
	−10	6.008	0.636	11%	-	-	-
	5	3.550	0.303	9%	3.445	0.147	4%
	20	0.510	0.063	12%	0.539	0.025	5%

The variation coefficients (CV) of the results obtained are around 10% for the PMB25 mixture and lower than 10% for the PMB50 mixture. Thus, the results obtained are valid since CV values lower than 10% are statistically considered very good.

The tensile strength values of PMB25 increased as the temperature decreased until a maximum value around 6.0 MPa was obtained at −10 °C. The tensile strength slightly decreased when the temperature was reduced to −20 °C, a result consistent with those from the literature [12]. This behavior can be explained by the event of low-temperature cracking when thermal tensile stresses exceed the fracture strength of asphalt mixtures [51]. A similar increase in the tensile strength can be observed for the PMB50 mixture while the temperature is reduced, although only two temperatures have been tested. Comparing both mixtures, PMB50 presented a slightly better behavior at 20 °C, but the opposite happened at 5 °C, with PMB25 presenting a higher tensile strength. Nevertheless, the tensile strength results of both mixtures are analogous at these test temperatures.

The failure strain results for PMB25 and PMB50 mixtures are presented in Figure 8. By analyzing the coefficient of variation values, the failure strain results can be considered excellent at −20 °C (CV of 9%), good to acceptable at −10 to 5 °C (CV values around 20%), and presented high variability at 20 °C (CV values up to 46%). Due to this disparity, the test temperature of 20 °C was no longer used in the subsequent tests presented in this work, and tests were mainly developed for low-temperature evaluation.

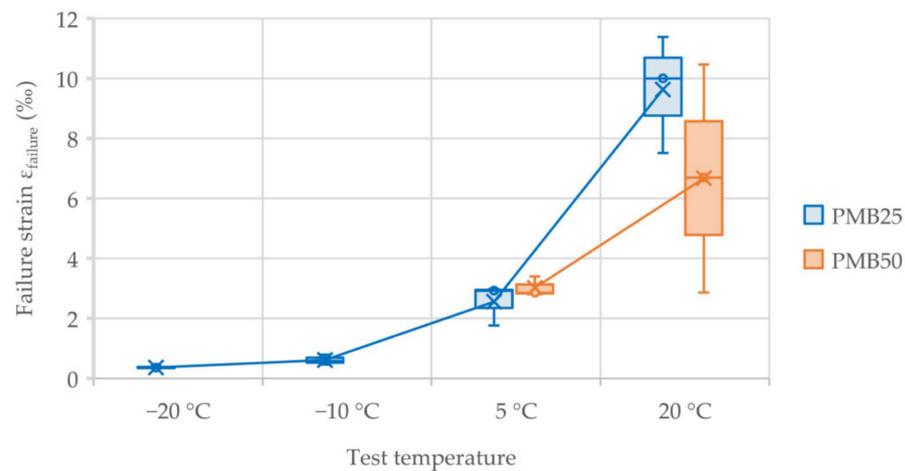


Figure 8. Failure strain results, $\epsilon_{failure}$, from the uniaxial tension stress test (UTST) for SMA 11 mixtures with PMB25 and PMB50.

Generally, the failure strain decreased with the test temperature reduction as the asphalt mixture becomes stiff. For the PMB25 mixture, the lowest test temperature ($-20\text{ }^{\circ}\text{C}$) presents a failure strain over twenty times shorter than the one measured at the highest test temperature ($20\text{ }^{\circ}\text{C}$). However, the failure strain change is low for temperatures between -10 to $-20\text{ }^{\circ}\text{C}$ once the mixtures have brittle behavior and reduced strain failure values.

The failure strain results of the PMB50 mixture presented a low variation at $5\text{ }^{\circ}\text{C}$ (CV of 9%) but a significant variation (CV of 46%) at $20\text{ }^{\circ}\text{C}$. The PMB50 mixture presented a slightly higher failure strain than PMB25, even though their average values are similar.

Considering the similar results from laboratory tests performed on the asphalt binders and the UTST tests performed on the asphalt mixtures, and given that PMB25 presented lower cost and behaved as reasonably as PMB50, the subsequent low-temperature mechanical tests were only performed in the PMB25 asphalt mixture.

3.4.2. Thermal Stress Restrained Specimen Test (TSRST)

The thermal stress restrained specimen test was performed according to the standard EN 12697-46 but using three different cooling rates: $\Delta T = -2, -5,$ and $-10\text{ }^{\circ}\text{C/h}$. Figure 9 shows the results from the various tests for each cooling rate, namely the cryogenic stress progression over the temperature $\sigma_{cry}(T)$ between 20 and $-20\text{ }^{\circ}\text{C}$. The specimens behaved as expected from the test principle.

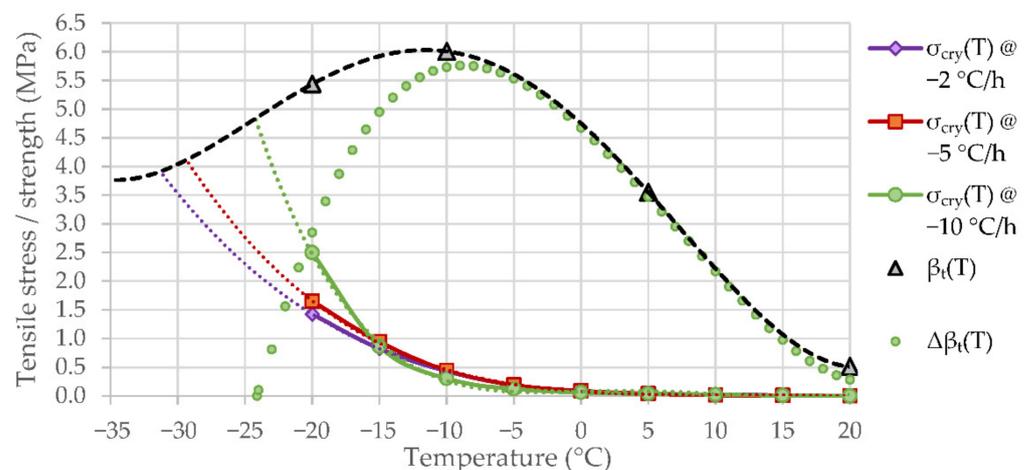


Figure 9. Mean results of the thermal stress restrained specimen test (TSRST) tests for PMB25 mixture at three different cooling rates.

The testing machine CS7400S could not maintain a stable cooling rate at temperatures below $-20\text{ }^{\circ}\text{C}$, which then restricts the evaluation of the failure stress $\sigma_{\text{cry, failure}}$ and the failure temperature T_{failure} of the studied mixture. However, the lab results in the range of 20 and $-20\text{ }^{\circ}\text{C}$ were perfectly adjusted with fourth-degree polynomial functions to estimate the failure stress and temperature. Simultaneously, the tensile strength variation with temperature, $\beta t(T)$, was obtained from the UTST results adjusting a cubic spline function (as mentioned in EN 12697-46 standard). Thus, the failure stress $\sigma_{\text{cry, failure}}$ and the failure temperature T_{failure} were estimated to occur when the failure stress $\sigma_{\text{cry}}(T)$ curve at each cooling rate meets the tensile strength $\beta t(T)$ diagram.

Figure 9 also shows the tension strength reserve $\Delta\beta t(T)$, which the mixture can support under external loads in addition to the low-temperature thermal stresses. This value is calculated as the difference between the tensile strength $\beta t(T)$ and the failure stress $\sigma_{\text{cry}}(T)$ curves and is only presented for the standard cooling rate of $-10\text{ }^{\circ}\text{C/h}$.

All cooling rates presented similar stresses between 20 and $-10\text{ }^{\circ}\text{C}$ in the TSRST tests. Below $-10\text{ }^{\circ}\text{C}$ and more visibly below $-15\text{ }^{\circ}\text{C}$, the differences of the three cooling rates become evident. The faster the cooling rate, the higher the thermal stresses the mixture will present at very cold temperatures (lower than $-15\text{ }^{\circ}\text{C}$), enabling thermal cracking. Thus, the correct definition of the cooling rate is vital in the design stage to accurately reflect the actual conditions observed in the pavement. Certain climate regions have more propensity for fast cooling events, which may also occur more often. Thus, the region’s climate of the pavement should be carefully studied in order to design the asphalt mixture properly.

The estimated failure stress $\sigma_{\text{cry, failure}}$ increased from 3.4 to 4.8 MPa , and the failure temperature T_{failure} increased from -32 to $-24\text{ }^{\circ}\text{C}$ when the cooling rate is augmented from -2 to $-10\text{ }^{\circ}\text{C/h}$. Thus, faster cooling rates result in asphalt mixture failure at higher stress levels and temperatures, which may occur more often. Higher failure temperatures from TSRST indicate that low-temperature cracking may increase [52]. Nevertheless, the results obtained are lower than $-24\text{ }^{\circ}\text{C}$, which is an excellent result since it is not predictable that lower temperatures will be observed in several climate regions.

The TSRST cooling rate suggested by EN 12697-46 standard is $-10\text{ }^{\circ}\text{C/h}$. This cooling rate subjects the SMA mixture to significantly higher stresses than the slower cooling rates for temperatures below $-15\text{ }^{\circ}\text{C}$ (e.g., tensile stress is 66% higher at $-20\text{ }^{\circ}\text{C}$). The tension strength reserve $\Delta\beta t(T)$ available to support external loads, in addition to the low-temperature thermal stresses, is at its maximum at $-10\text{ }^{\circ}\text{C}$ and becomes critically low for temperatures below $-22\text{ }^{\circ}\text{C}$, at which point the mixture can easily crack under external loads from traffic.

3.4.3. Relaxation Test (RT)

The initial input values for the relaxation test are shown in Table 7. The initial imposed stress derives from the UTST and should not be greater than 75% of the obtained tensile strength βt . Based on that value, an input strain was calculated and imposed in the specimen during the test. The relaxation time is obtained when the tensile stress (resulting from applying the strain) is reduced to $36.8 \pm 0.1\%$ of its initial value.

Table 7. Input values and results for the relaxation test of the PMB25 mixture.

Parameter		$-20\text{ }^{\circ}\text{C}$	$-10\text{ }^{\circ}\text{C}$	$5\text{ }^{\circ}\text{C}$
Input value	75% peak stress from UTST (MPa)	-4.078	-4.506	-2.663
	Corresponding input strain (‰)	0.30	0.48	0.93
Final result	Actual imposed stress (MPa)	-3.902	-4.399	-2.645
	Relaxation stress (MPa)	-1.436 (not reached)	-1.619	-0.973
	Relaxation time t_{rel} (s)	Not reached	2206	47
	Test stopping time (s)	$172,800$	2206	47
	Remaining tensile stress σ_{rem} (MPa)	-1.625	-1.619	-0.973

Relaxation modulus is a main viscoelastic parameter of an asphalt mixture and is an essential basis for evaluating and analyzing the performance and for predicting long-term pavement stability [53]. The evolution of stress versus time measured in the relaxation tests for the PMB25 mixture at three test temperatures (5, −10, and −20 °C) is shown in Figure 10. The corresponding final results were also presented in Table 7. During the tests, all specimens behaved as anticipated in the standard procedure.

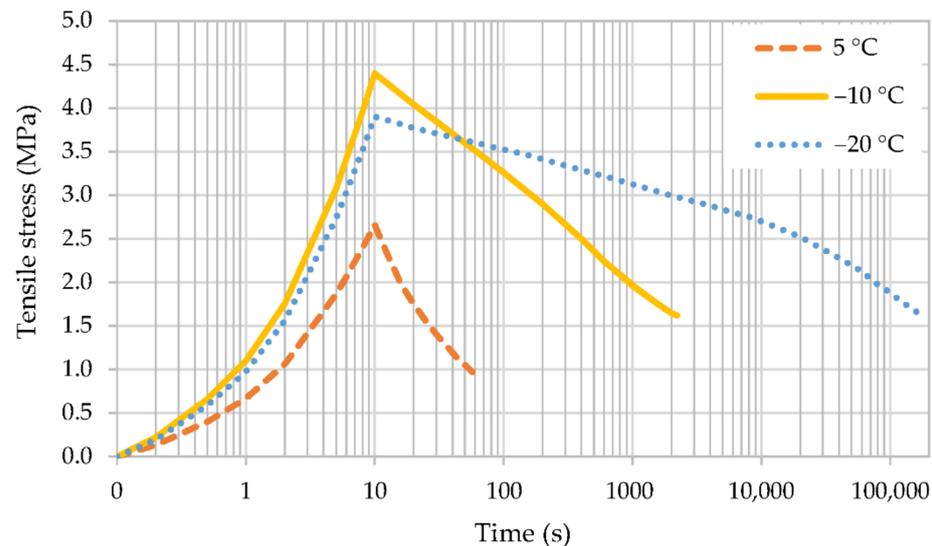


Figure 10. Evolution of the tensile stress over time for the PMB25 mixture in the relaxation test at different temperatures.

The asphalt mixture reached the relaxation time very quickly at 5 °C, i.e., under a minute. The asphalt mixtures’ rheological properties are affected by temperature, vehicle load, and vehicle speed. Once pavements are subjected to stress, the stress is gradually dissipated over time, and relaxation occurs. When the temperature is high, the pavement’s stress dissipates quickly because of a better relaxation ability, as verified in this study.

The relaxation time was reached after approximately 36 min at −10 °C. However, the mixture could not reach the relaxation time at −20 °C during the 48 h proposed by EN 12697-46. This test should be concluded when the stress level falls below 36% of its initial value (1.436 MPa at −20 °C). The remaining tensile stress σ_{rem} after the test at −20 °C has ended was 1.625 MPa, near the pre-established relaxation stress of 36%.

Overall, the studied mixture presents an excellent relaxation behavior at low (−10 °C) and medium (5 °C) temperatures. Some slow relaxation issues of the mixture may only occur at very low temperatures (−20 °C) if the pavement is exposed to these conditions for long periods.

Other studies observed that the pavement stress dissipates slowly because of poor relaxation ability when its temperature is very low [53], confirming the current study’s results. When this occurs, cracks in the pavement may appear since the final stress are more significant than the mixture’s ultimate tensile strength.

3.4.4. Tensile Creep Test (TCT)

A creep test (sometimes referred to as a stress-relaxation test) is used to determine the amount of deformation a material experiences over time while under a continuous tensile load at a constant temperature. The results from the tensile creep tests carried out at different temperatures are shown in Figure 11, which result from the application of the tensile stresses recommended in EN 12697-46, as presented in Table 2.

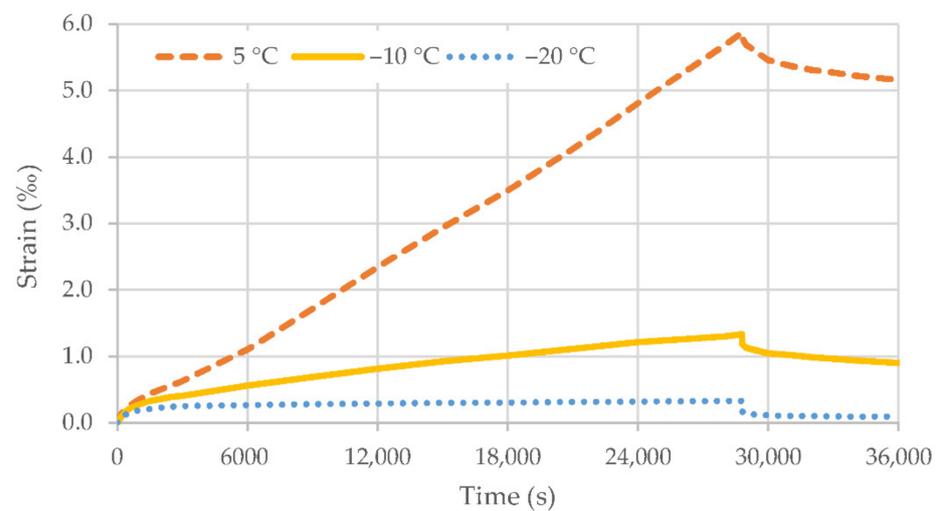


Figure 11. Results of tensile creep tests on the PMB25 mixture at different temperatures.

At the limit temperature of $-20\text{ }^{\circ}\text{C}$, the creep test resulted in a low strain value during the 8 h loading period. Then, it recovered a significant part of that strain during the following 2 h rest period.

As expected, the maximum strain value obtained at $-10\text{ }^{\circ}\text{C}$ was higher than that observed in the previous temperature, recovering a significant part of its maximum strain. Its peak strain level was $1.33\text{ }‰$ and recovered $0.43\text{ }‰$, which presents an approximate recovery of 30% of the maximum strain. The stress level defined in the standard for this temperature caused a worse behavior of the mixture.

Nevertheless, the worst performance was observed at $5\text{ }^{\circ}\text{C}$, with a considerable deformation under the stress level defined in the standard, and only a small proportion of that deformation was recovered. The mixture reached a maximum strain level of $5.79\text{ }‰$ and recovered $0.63\text{ }‰$, which results in a recovery of only 11% of the maximum deformation.

These results indicate that this asphalt mixture most likely will not present deformation issues at very low temperatures. The same cannot be said for the intermediate temperatures, particularly at $5\text{ }^{\circ}\text{C}$, considering the stress values defined in the standard. The mixture only recovered 11% of the maximum deformation in this temperature range, indicating that deformation can occur during the pavement’s lifetime, depending on the actual loading conditions.

This test simulates the effects caused in the pavement when a load is applied by vehicles or thermal stresses and then is removed when those conditions change. If the asphalt mixture cannot recover from the load promptly, this can result in deformation damage to the pavement as more loads are applied. These effects of transitory loading and unloading of asphalt mixtures cause deformations which can, over time, result in rutting or cracking of the pavement [54].

When modeling this test, Horny et al. [55] observed that the viscoelastic strains calculated with the model were more extensive than the experimental ones obtained during the creep’s unloading phase test. Pszczoła and Judycki [56] also observed that the elastic modulus and viscosity coefficient increased with the decrease in temperature used in the TCT due to the increased stiffness of the asphalt mixtures.

3.4.5. Uniaxial Cyclic Tensile Stress Test (UCTST)

Table 8 presents the input values for the base stress $\sigma_{cry}(T)$, which is the expected cryogenic stress derived from the TSRST test, the stress caused by the traffic load $\Delta\sigma$, and the peak stress σ_{tot} (the sum of the previous one) used in the uniaxial cyclic tensile stress test. These values intend to reflect the cyclic tension/compression loading patterns observed at the bottom of the asphalt pavement layer [57]. A summary of the final results obtained from the UCTST tests is also shown in Table 8.

Table 8. Input values and results for the uniaxial cyclic tensile stress test of PMB25 mixture.

Parameter		−20 °C	−10 °C	5 °C
Input value	Base stress $\sigma_{cry}(T)$ @ −10 °C/h (MPa)	−2.50	−0.30	−0.04
	Traffic load $\Delta\sigma$ (MPa)	−1.60	−1.60	−1.60
	Peak stress σ_{tot} (MPa)	−4.10	−1.90	−1.64
Final result	Type of failure	Not reached	Not reached	Fracture
	Number of load applications, $N_{failure}$	>2.0 million	>2.0 million	12,500

A criterion of a maximum of 2.0 million cycles was established for the specimen to reach failure. This criterion was based on a restriction of time for the laboratory tests to be performed. It results in continuous testing for about 2.3 days.

The specimens tested at the highest temperature used (5 °C) reached the expected failure. The mixture showed an almost linear progressive curve of the strains recorded until failure after 12,500 cycles, on average. This result is consistent with the higher creep deformation of the SMA mixture at 5 °C in the TCT test.

The SMA mixture was also tested at −10 and −20 °C without reaching failure after the pre-established 2.0 million cycles criterion. Nevertheless, the mixture presented a linear progressive strain behavior at both temperatures. Although the asphalt mixtures are stiffer and brittle at these temperatures, they also suggest a more elastic behavior, which justifies the low accumulations of strains and the absence of failure due to cyclic loading under these conditions.

4. Conclusions

This work focused on evaluating the influence of asphalt binder modification with 2.5% and 5.0% SBS in SMA mixtures' low-temperature performance. Several laboratory tests characterized the asphalt binders to obtain the tensile strength properties at cold temperatures. The main conclusions drawn from this study are as follows:

- The modification of bitumen with SBS increased the stiffness, reducing the penetration and increasing the softening point temperature.
- The PMBs presented an excellent elastic recovery and force-ductility performance.
- The tensile strength of the SMA mixtures achieved its maximum value at −10 °C. Then, the tensile strength slightly decreased when the temperature was further reduced.
- The failure strain decreased with the test temperature reduction as the asphalt mixture becomes brittle. At the lowest tested temperature, the SMA presents a failure strain over twenty times shorter than the one measured at 20 °C.
- PMB25 was selected for the remainder of this work. Its lower cost and similar results between both asphalt binders, including in the UTST tests, justified this selection.
- The TSRST thermal failure was estimated to occur at temperatures lower than −24 °C, although the stress reserve is small below −22 °C.
- The SMA mixture presents an excellent relaxation behavior at temperatures equal to or higher than −10 °C, but some slow relaxation issues may occur at very low temperatures (−20 °C).
- The TCT test results indicate that SMA mixture is not likely to present creep deformation issues at very low temperatures, but the same cannot be said at 5 °C.
- The low accumulations of strains at −10 and −20 °C justify the absence of failure due to cyclic loading (UCTST) under these conditions, unlike the SMA mixture's failure after 12,500 cycles when tested at 5 °C.

There are some perspectives or future works planned succeeding this work. The low-temperature rheological properties of the asphalt binders evaluated in the DSR and BBR should be related to the asphalt mixtures' low-temperature performance. Other types of binders and mixtures with different compositions should also be evaluated to determine the main factors that control low-temperature performance. The effect of aging on the low-temperature performance of asphalt binders and mixtures should also be studied.

Finally, in situ tests in a cold climate experimental trial would be essential to determine the laboratory tests' relation with actual pavement low-temperature performance.

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