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Multiscale mechanical characterization and constitutive modeling of a modified bituminous composite: Linking laboratory performance to time-temperature dependent and multi-model prediction

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Abstract

The development of high-performance asphalt mixtures capable of simultaneously improving resistance to rutting, fatigue cracking, moisture damage, and temperature-induced degradation remains a major challenge in pavement engineering. This study investigates the mechanical performance and thermo-viscoelastic behavior of a modified asphalt mixture incorporating 0.3% CECABASE 300, a bio-based adhesion and workability-enhancing additive, compared with an unmodified reference mixture. A comprehensive experimental program was conducted on a 35/50 penetration grade bitumen and asphalt mixture, including binder characterization, Marshall properties, gyratory compaction, water sensitivity, rutting

resistance, stiffness modulus, and fatigue performance. The results demonstrated that the incorporation of CECABASE 300 improved binder consistency, increased softening temperature, enhanced compaction behavior, and significantly improved moisture resistance and mechanical performance. The modified mixture exhibited higher stiffness, improved resistance to permanent deformation, and enhanced fatigue performance, confirming the beneficial effect of the additive on the overall pavement response. The linear viscoelastic behavior of the modified mixture was further investigated through complex modulus testing over a wide frequency range and temperature conditions. The time-temperature superposition principle combined with the Arrhenius shift function was applied to construct the master curve at a reference temperature of 15 °C. The experimental master curve was subsequently modeled using three advanced constitutive approaches: the Huet–Sayegh model, the 2S2P1D model, and a 39-parameter Generalized Kelvin–Voigt model (GKVM). The optimized models accurately reproduced the experimental master curve, with prediction errors below 0.5%. Among the investigated formulations, the GKVM provided the highest accuracy, with the lowest root mean square error (121.22 MPa), due to its ability to represent the broad relaxation spectrum through multiple Kelvin elements. The 2S2P1D model provided an effective compromise between accuracy and physical interpretation, while the Huet–Sayegh model demonstrated satisfactory predictive capability with reduced parameter complexity. The combined experimental and modeling results demonstrate that CECABASE 300-modified asphalt mixtures exhibit enhanced mechanical performance and favorable thermo-viscoelastic characteristics, making them promising materials for durable pavement wearing courses. The calibrated constitutive models provide reliable tools for advanced pavement design and numerical simulation under complex loading and environmental conditions.

Keywords: Modified asphalt mixture; CECABASE 300; Wearing course; Mechanical performance; Time–temperature superposition principle; Master curve; Thermo-viscoelastic modeling; Generalized Kelvin–Voigt model.

1. Introduction

The continuous increase in traffic volume, axle loads, and environmental stresses has imposed increasingly severe requirements on asphalt pavement materials. The wearing course, as the uppermost structural layer directly exposed to traffic and climatic actions, plays a crucial role in pavement durability, safety, and service life. Conventional asphalt mixtures frequently experience premature deterioration mechanisms, including permanent deformation at high temperatures, fatigue cracking under repeated traffic loading, moisture-induced damage, and thermal cracking under low-temperature conditions. Consequently, the development of innovative asphalt materials combining enhanced mechanical performance and improved long-term durability has become a major research objective in pavement engineering. The performance of asphalt mixtures is strongly governed by the thermo-viscoelastic behavior of the bituminous binder and the interactions between binder, aggregates, and additives. Under traffic loading, asphalt materials exhibit a complex response that varies significantly with temperature and loading frequency, evolving from an almost elastic behavior at low temperatures or high loading frequencies to a viscous response at high temperatures or long loading durations. Therefore, accurate characterization and modeling of asphalt viscoelastic properties are essential for predicting pavement performance and implementing reliable mechanistic empirical design approaches.

In recent years, the incorporation of chemical additives, warm-mix technologies, adhesion promoters, and bio-based modifiers has attracted considerable attention as a strategy to improve asphalt mixture performance while reducing energy consumption and environmental impacts. Among these technologies, CECABASE 300 has demonstrated potential as a bio-

based additive capable of improving binder–aggregate adhesion, mixture workability, and compactability. However, although several studies have investigated the influence of additives on conventional mechanical properties, a comprehensive understanding of their effects on the thermo-viscoelastic response and constitutive behavior of asphalt mixtures remains limited. The complex modulus is one of the most representative parameters for describing the linear viscoelastic behavior of asphalt mixtures. Since laboratory measurements can only be performed within a limited range of temperatures and frequencies, the time–temperature superposition principle (TTSP) is commonly applied to construct a master curve describing material stiffness over a wide range of loading conditions. The accuracy of the resulting master curve depends strongly on the validity of the temperature shift function and the ability of the selected constitutive model to reproduce the experimental behavior.

Several rheological models have been proposed to describe the viscoelastic response of asphalt materials. The Huet–Sayegh model, based on fractional derivatives, has been widely used because of its simplicity and ability to reproduce the sigmoidal shape of asphalt mixture master curves with a limited number of parameters. The 2S2P1D model extends this approach by introducing two springs, two parabolic elements, and one dashpot, providing an improved description of viscous flow and long-term relaxation mechanisms. More recently, Generalized Kelvin–Voigt models have been employed to represent the relaxation spectrum through multiple discrete Kelvin elements, offering increased flexibility and accuracy for describing complex viscoelastic behavior. However, the comparison between these models using experimentally validated master curves for modified asphalt mixtures remains insufficiently investigated. A comprehensive understanding of the relationship between laboratory performance indicators and thermo-viscoelastic constitutive parameters is essential for developing reliable pavement materials. In particular, linking mechanical properties such as

moisture resistance, rutting resistance, stiffness modulus, and fatigue behavior with advanced rheological modeling can provide deeper insight into the mechanisms controlling pavement durability.

Therefore, the objective of this study is to investigate the effect of CECABASE 300 incorporation on the mechanical and thermo-viscoelastic properties of a 35/50 penetration grade bitumen asphalt mixture designed for wearing courses. A complete experimental program was conducted, including binder characterization, Marshall properties, gyratory compaction, Duriez water sensitivity, indirect tensile strength ratio, rutting resistance, stiffness modulus, fatigue testing and viscoelastic modeling by constitutives models : Huet–Sayegh, 2S2P1D and Generalized Kelvin–Voigt.

2. Literature Review

Asphalt pavements constitute one of the most widely used transportation infrastructures worldwide due to their relatively low construction cost, ease of maintenance, high riding comfort, and adaptability to varying traffic and environmental conditions. The long-term performance of asphalt pavements, however, depends on the complex interaction between material properties, traffic loading, climatic conditions, construction practices, and pavement structural design. Increasing traffic volumes, heavier axle loads, and more severe environmental conditions have significantly accelerated pavement deterioration, creating an urgent need for asphalt mixtures with improved durability, enhanced mechanical performance, and longer service lives [1,2]. Asphalt mixtures are heterogeneous composite materials consisting primarily of mineral aggregates, bituminous binder, air voids, and, increasingly, performance-enhancing additives. Unlike purely elastic engineering materials, asphalt mixtures exhibit highly complex thermo-viscoelastic behaviour resulting from the intrinsic properties of the bituminous binder and its interaction with the aggregate skeleton. Consequently, their mechanical response depends simultaneously on stress level, loading

duration, loading frequency, and temperature. This time-dependent behaviour makes asphalt materials particularly challenging to characterize and model using conventional elastic theories [3-5]. The bituminous binder is the most influential component governing the rheological behaviour of asphalt mixtures. Bitumen possesses a colloidal microstructure composed of asphaltenes dispersed within a maltene phase, whose balance largely determines stiffness, viscosity, elasticity, durability, and resistance to deformation. Changes in this colloidal equilibrium, whether through oxidation, aging, or chemical modification, directly influence pavement performance throughout its service life [5].

One of the major challenges facing pavement engineers is the progressive aging of asphalt binders. During production, construction, and in-service exposure, oxidation and volatilization gradually increase binder stiffness while reducing ductility and relaxation capacity. Although moderate hardening may initially improve rutting resistance, excessive aging significantly increases brittleness, leading to fatigue and thermal cracking that shorten pavement service life [6]. The interaction between aging mechanisms and mixture composition has consequently become a major area of research aimed at improving pavement durability. To address these limitations, considerable attention has been devoted to asphalt modification through the incorporation of performance-enhancing additives. Polymer modification has emerged as one of the most effective approaches for improving elasticity, temperature susceptibility, fatigue resistance, rutting resistance, and overall durability of asphalt binders. Polymers modify the internal structure of bitumen by altering the interaction between the dispersed and continuous phases, thereby improving the rheological response across a broad range of service temperatures [7,8]. The effectiveness of these modifiers depends not only on their chemical composition but also on compatibility with the base binder and the resulting microstructural evolution during mixing and aging [5].

Alongside polymer modification, increasing environmental awareness has stimulated the development of sustainable asphalt technologies. Warm Mix Asphalt (WMA) technologies reduce production and compaction temperatures while maintaining acceptable workability and mechanical performance. Lower production temperatures decrease fuel consumption, greenhouse gas emissions, and binder aging during manufacture, thereby contributing to more sustainable pavement construction practices [9,10]. The incorporation of chemical, organic, and foaming additives has therefore become an integral component of modern asphalt engineering, balancing performance enhancement with environmental objectives.

The engineering performance of asphalt mixtures is commonly evaluated through several key mechanical properties, including stiffness, rutting resistance, fatigue resistance, moisture susceptibility, and long-term durability. Adequate binder film thickness has been shown to play a critical role in protecting the binder against oxidative aging while ensuring sufficient cohesion and adhesion within the mixture [11]. Consequently, mixture design methodologies increasingly emphasize the simultaneous optimization of volumetric properties and mechanical performance [1]. Modern pavement engineering has progressively shifted from empirical design approaches toward mechanistic–empirical methodologies that incorporate fundamental material properties into pavement analysis and structural design. Rather than relying solely on empirical relationships, mechanistic approaches use constitutive models capable of describing the stress–strain response of asphalt mixtures under realistic loading conditions. Dynamic modulus testing has consequently become one of the most important laboratory methods for characterizing asphalt stiffness over a wide range of temperatures and loading frequencies [12,13]. These experimentally determined properties provide essential inputs for pavement performance prediction models and finite element simulations [14]. Classical rheological models, including the Maxwell and Kelvin–Voigt formulations, provide the theoretical foundation for describing creep, relaxation, and stress redistribution in

viscoelastic materials [3,4]. However, the complex behaviour of asphalt mixtures often exceeds the predictive capabilities of these simple mechanical analogues, motivating the development of generalized and fractional viscoelastic models that more accurately reproduce observed material responses over broad loading spectra [15].

A key concept enabling the characterization of asphalt viscoelasticity is the Time–Temperature Superposition Principle (TTSP). This principle recognizes that the influence of temperature on viscoelastic response can be represented by horizontal shifts of stiffness curves measured at different temperatures, allowing the construction of master curves that characterize material behaviour over loading times far beyond practical laboratory testing. The Williams–Landel–Ferry (WLF) equation and the Arrhenius relationship provide the theoretical basis for determining temperature-dependent shift factors, depending on the thermorheological characteristics of the material [16-18]. Building upon these theoretical foundations, several advanced constitutive models have been proposed specifically for asphalt materials. Among these, the Huet–Sayegh model represents one of the earliest and most influential rheological formulations capable of describing the linear viscoelastic behaviour of bituminous materials across wide frequency and temperature ranges [19-20]. Subsequent developments culminated in the 2S2P1D model, which provides an improved mathematical representation linking the rheological behaviour of bituminous binders and asphalt mixtures while maintaining a physically meaningful interpretation of model parameters [21-24]. These models have become widely adopted in pavement research because of their ability to accurately reproduce dynamic modulus master curves and facilitate numerical implementation within mechanistic pavement analysis.

The increasing availability of advanced computational tools has further enhanced the analysis of asphalt pavements through finite element modelling. Viscoelastic constitutive models can be incorporated into numerical simulations to predict pavement responses under realistic

traffic loading conditions, enabling engineers to evaluate stress distribution, strain accumulation, fatigue damage, and rutting progression with greater accuracy. Such approaches provide a powerful framework for integrating laboratory characterization with structural pavement analysis and performance prediction [25-29]. Within this broader scientific context, asphalt modification through performance-enhancing additives has become an increasingly important research area. Additives are expected not only to improve conventional engineering properties but also to modify the viscoelastic response of asphalt mixtures, influencing stiffness evolution, temperature susceptibility, fatigue resistance, permanent deformation, and long-term durability. Understanding these interactions requires integrating knowledge from binder chemistry, mixture design, rheological characterization, constitutive modelling, and numerical simulation. Such an integrated approach is essential for developing asphalt materials capable of meeting the increasing demands of modern transportation infrastructure while simultaneously addressing sustainability and lifecycle performance objectives. Accordingly, this literature review is organized to provide a progressive examination of the scientific foundations underpinning additive-enhanced asphalt mixtures. The review first examines the chemistry of bitumen, aging mechanisms, and asphalt modification technologies. It then discusses the mechanical behaviour of asphalt mixtures and the principles of linear viscoelasticity before reviewing dynamic modulus characterization and the Time–Temperature Superposition Principle. Subsequently, advanced rheological models, including the Huet–Sayegh and 2S2P1D formulations, are presented together with generalized viscoelastic models relevant to asphalt materials. Finally, the application of these constitutive relationships in finite element modelling is examined, providing the theoretical framework that supports the investigation of additive-enhanced asphalt mixtures undertaken in this research.

2.1 Bitumen Chemistry, Asphalt Mixture Behaviour, and Modification

Bituminous materials have been employed in road construction for more than a century owing to their excellent adhesive properties, waterproofing capability, flexibility, and ability to distribute traffic-induced stresses. Modern asphalt pavements continue to dominate highway infrastructure because they provide an effective balance between structural performance, construction efficiency, maintenance flexibility, and economic viability. Despite these advantages, increasing traffic volumes, higher axle loads, and more severe climatic conditions have intensified the demand for asphalt mixtures with enhanced durability and mechanical performance. Consequently, significant research has focused on understanding the physicochemical behaviour of bitumen and on developing modified asphalt mixtures capable of maintaining satisfactory performance throughout their design life [1,2]. An asphalt mixture is a multi-phase composite material consisting of mineral aggregates, bituminous binder, air voids, and, in many modern applications, performance-enhancing additives. The aggregate skeleton provides the primary load-bearing framework and contributes significantly to strength, stiffness, and resistance to permanent deformation. The addition of modifiers further influences the rheological properties of the binder and, consequently, the overall mechanical response of the asphalt mixture [1]. The response of asphalt mixtures to applied loading depends not only on the magnitude of stress but also on loading duration, loading frequency, and temperature. At low temperatures or under rapid loading, asphalt mixtures behave relatively stiffly, approaching elastic behaviour. Conversely, at elevated temperatures or under sustained loading, the viscous characteristics of the binder become increasingly dominant, resulting in greater permanent deformation and stress relaxation [3,4]. Consequently, understanding the chemistry of bitumen is fundamental to explaining the engineering

behaviour of asphalt mixtures under varying service conditions [5]. Oxidation, volatilization of lighter components, ultraviolet radiation, moisture, and repeated traffic loading gradually increase binder stiffness while reducing flexibility and relaxation capacity. Although a moderate increase in stiffness may improve resistance to rutting during the early stages of pavement life, continued aging generally increases susceptibility to fatigue cracking and thermal cracking, ultimately reducing pavement durability [6].

Depending on their composition and mechanism of action, additives may alter the chemical interactions within the binder, reinforce the aggregate–binder interface, or modify the internal rheological structure responsible for viscoelastic behaviour [7,8]. Nevertheless, the effectiveness of polymer modification depends strongly on polymer type, dosage, compatibility with the base binder, storage stability, and aging characteristics, making a comprehensive understanding of binder chemistry essential for optimizing modified asphalt formulations [5,8]. The incorporation of chemical additives, organic waxes, or foaming technologies improves workability at lower temperatures, thereby reducing fuel consumption, emissions, and occupational exposure while contributing to more sustainable pavement construction practices [9,10]. The increasing diversity of available asphalt additives has expanded the possibilities for tailoring pavement materials to specific service conditions. Bitumen aging progressively alters the balance among the SARA fractions, leading to significant changes in both chemical composition and rheological properties. According to [6], oxidative aging converts lighter molecular constituents into increasingly polar compounds through a series of oxidation reactions. This process is generally characterized by a reduction in aromatic content, an increase in resin concentration, the gradual formation of larger asphaltene-like molecules, and enhanced molecular association. A substantial body of research on the modification of bitumen through the incorporation of additives has also been

published worldwide [30-49]. This aspect will not be discussed in detail here, as this section is already quite extensive.

2.2 Mechanical Response of Asphalt Mixtures

The mechanical behaviour of asphalt mixtures is governed by the interaction between aggregate structure, bituminous binder properties, volumetric characteristics, environmental conditions, and applied loading. Unlike purely elastic materials, asphalt mixtures exhibit a combination of elastic, viscous, and plastic responses, resulting in a complex behaviour commonly described as thermo-viscoelastic-plastic. The long-term performance of asphalt pavements is primarily governed by the ability of asphalt mixtures to withstand the major mechanisms of deterioration encountered during service. These include permanent deformation (rutting), fatigue cracking caused by repeated traffic loading, thermal cracking resulting from temperature fluctuations, moisture-induced damage associated with the loss of adhesion between the binder and aggregates, and aging-related embrittlement arising from the progressive oxidation of the asphalt binder. According to [1], successful asphalt mixture design requires balancing stiffness, flexibility, durability, and workability rather than maximizing a single mechanical property.

2.2.1 Asphalt Mixture as a Viscoelastic Composite

The fundamental characteristic distinguishing asphalt mixtures from many traditional construction materials is their viscoelastic response. A general stress–strain relationship can be represented as:

$$\sigma(t) = \int_0^t E(t-\tau)\frac{d\varepsilon(\tau)}{d\tau}d\tau \qquad (1)$$

where: σ(t) = stress response, ε(t) = strain history, E(t) = relaxation modulus and t = time.

This relationship, based on the Boltzmann superposition principle [50,51], indicates that the current response depends on the complete loading history rather than only the current stress

state. For asphalt mixtures, the viscoelastic nature of the material implies that the mechanical response depends not only on the magnitude of the applied load but also on temperature and loading time.

Rutting is one of the dominant failure mechanisms (Fig. 1-a) in asphalt pavements exposed to high temperatures and heavy traffic loading. Permanent deformation (rutting) develops as a result of several interacting mechanisms, including the viscous flow of the bituminous binder, the rearrangement of aggregate particles under repeated loading, inadequate mixture stiffness, and insufficient resistance to shear stresses. Consequently, the rutting susceptibility of asphalt mixtures is governed by both the rheological properties of the binder and the structural characteristics of the aggregate skeleton. Brown et al., [2] emphasized that rutting resistance depends on the combined effects of binder properties and mixture composition. Key influencing factors include binder viscosity, the degree of aggregate interlock, air void content, and the use of performance-enhancing additives or modifiers.

Fatigue cracking results (Fig. 1-b) from repeated traffic loading that produces progressive damage accumulation. Unlike rutting, fatigue failure is associated with repeated tensile strains, crack initiation and crack propagation. The fatigue performance of asphalt mixtures depends strongly on binder flexibility, aging state, mixture stiffness and crack resistance. Modified binders can enhance fatigue performance through improved elasticity, delayed crack propagation and improved energy dissipation capability. Zhu et al. [8] highlighted that polymer modification changes the viscoelastic balance of the binder by increasing the elastic component while maintaining sufficient viscous relaxation. Kandhal and Chakraborty [11] demonstrated that binder film thickness significantly influences asphalt mixture durability. A simplified fatigue relationship is often expressed as:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \tag{2}$$

where: N_f = number of load cycles to failure ; ε_t = tensile strain ; k_1, k_2 = experimentally determined coefficients.

2.2.2 Performance-based characterization

Traditional asphalt mixture design relied heavily on empirical indicators such as Marshall stability and flow. Modern approaches increasingly emphasize fundamental performance indicators given by dynamic modulus. The dynamic modulus describes mixture stiffness under sinusoidal loading and provides essential information for mechanistic pavement design. Pellinen et al. [12] and [13] demonstrated that dynamic modulus measurements (Fig. 2) allow prediction of mixture response over broad temperature and frequency ranges. The dynamic modulus is defined as:

$$\left|E^*\right| = \frac{\sigma_0}{\varepsilon_0} \tag{3}$$

where: $|E*|$ = dynamic modulus, σ_0 = stress amplitude and ε_0 = strain amplitude.

2.3 Relationship between mechanical behaviour and constitutive modelling

For pavement materials, this behaviour explains why the same asphalt mixture can behave as a stiff elastic solid under rapid vehicle loading at low temperature but as a softer viscoelastic material under slow loading or elevated temperature. The theoretical foundation of this behaviour is provided by the theory of linear viscoelasticity developed for polymers and extended to asphalt materials by researchers such as [3,4,23]. Among these methods, the dynamic modulus test and the resulting master curve methodology have become fundamental tools for describing asphalt mixture stiffness over a broad range of loading conditions [12,13]. The dynamic modulus provides a quantitative representation of the mixture response under sinusoidal loading, while the master curve combines experimental results obtained at different

temperatures and frequencies into a single continuous relationship describing material behaviour over an extended time scale. Under sinusoidal loading (Fig. 3), stress and strain in a viscoelastic material are expressed as:

$$\sigma(t) = \sigma_0 \sin(\omega t) \text{ and } \varepsilon(t) = \varepsilon_0 \sin(\omega t - \delta) \quad (4)$$

where: σ_0 = stress amplitude, ε_0 = strain amplitude, ω = angular frequency and δ = phase angle.

2.4 Time–Temperature Superposition Principle (TTSP), WLF Equation, and Arrhenius Temperature Shift Models

Asphalt mixtures experience a wide range of temperatures and loading durations during their service life. A vehicle travelling at highway speed applies a short-duration load, whereas slow-moving traffic or standing loads produce much longer loading periods. Similarly, pavement temperatures can vary from extremely low winter conditions to high summer temperatures. Because asphalt materials are thermorheologically complex, directly measuring their mechanical response over the complete range of possible service conditions is practically impossible. A laboratory experiment covering decades of loading time would require unrealistic testing durations. To overcome this limitation, the Time–Temperature Superposition Principle (TTSP) has become one of the most important concepts in asphalt material characterization. The time–temperature superposition principle (TTSP) is based on the observation that temperature and loading time produce analogous effects on molecular mobility within viscoelastic materials. The theoretical basis of TTSP originates from polymer rheology [16] and has subsequently been extensively adopted for the characterization and modelling of asphalt materials [23].

The applicability of TTSP depends on the assumption of thermorheological simplicity. A material is considered thermorheologically simple when temperature changes affect all

relaxation mechanisms proportionally, without changing the fundamental shape of the relaxation spectrum. Mathematically:

$$E(t, T)=E(t/a_T, Tr) \quad (5)$$

where: E(t, T) = relaxation modulus at temperature T, $E(t, T_r)$ = relaxation modulus at reference temperature and a_T = temperature shift factor.

The shift factor represents the horizontal displacement required to superimpose data measured at different temperatures. For frequency-domain analysis:

$$f_r = f \times a_T \quad (6)$$

where: f_r = reduced frequency, and f = experimental frequency.

The reduced frequency represents the equivalent frequency at the selected reference temperature.

2.4.1 Temperature shift factors

The shift factor a_T is the key parameter controlling master curve construction. It can be determined by empirical fitting, WLF equation or Arrhenius equation. The appropriate model depends on temperature range and material characteristics. The WLF equation was developed by Williams, Landel, and Ferry [17] to describe temperature dependence near the glass transition region of amorphous polymers.The equation is:

$$\log\left(a_T\right) = -\frac{C_1\left(T - T_r\right)}{C_2 + \left(T - T_r\right)} \quad (7)$$

where: a_T = temperature shift factor, T = testing temperature, T_r = reference temperature, and C_1, C_2 = empirical constants.

The Arrhenius equation provides an alternative approach for describing temperature dependency. The general relationship is:

$$a_T\left(T, T_r\right) = \exp\left[\frac{E_a}{R}\left(\frac{1}{T} - \frac{1}{T_r}\right)\right] \quad (8)$$

where: E_a = activation energy =50 kcal/mole, R = universal gas constant, T = absolute temperature, and T_r = reference temperature.

Table 1 presents a comparison between the WLF and Arrhenius models, highlighting their respective applicability, advantages, and limitations for describing the temperature dependence of asphalt materials.

By evaluating changes in shift factors, master curve characteristics, and relaxation behavior, TTSP analysis can help quantify the influence of additives on the time- and temperature-dependent performance of asphalt binders and mixtures. This understanding forms the basis for the next parts addressing advanced rheological constitutive models, beginning with the Huet–Sayegh model, followed by the 2S2P1D model and generalized viscoelastic formulations. The time-temperature superposition principle provides the theoretical basis for predicting asphalt behaviour over extended ranges of loading frequency and temperature. Ferry [16] established the fundamental framework of TTSP, while Williams et al. [17] developed the WLF equation to describe temperature-dependent relaxation mechanisms. For asphalt mixtures, Di Benedetto et al. [23], Yussoff et al., [52] demonstrated the applicability of these concepts to bituminous materials from binder to mixture scale. Loulizi et al. [44] confirmed experimentally that asphalt mixture stiffness is strongly dependent on temperature and loading frequency. Alternatively, the Arrhenius relationship [18] provides an activation energy-based approach for describing temperature shift behaviour, particularly for materials exhibiting different thermo-rheological characteristics.

2.4.2 Viscoelastic Models

The Huet model, originally introduced by Huet [19] and subsequently extended by Sayegh [20], represents one of the earliest constitutive approaches specifically developed for describing the rheological behavior of bituminous materials (Table 2).

where: $E^*(\omega)$ = complex modulus, E_0 = equilibrium modulus, E_∞ = glassy modulus, ω = angular frequency, τ = characteristic time, h and k = fractional exponents, δ = weighting parameter.

The resulting Huet–Sayegh model has become a fundamental contribution to asphalt rheology, as it provides a compact mathematical framework for representing the complex modulus of asphalt binders and mixtures over a wide range of frequencies and temperatures. The Huet model was initially proposed by Huet [19] to represent the rheological response of bituminous materials using fractional elements. It was later extended by Sayegh [20] for asphalt mixtures, becoming one of the reference constitutive models in French pavement engineering. The Huet–Sayegh model (Table 2) is implemented in several pavement design tools and is widely used in the French mechanistic design method. The Huet–Sayegh model provides an essential link between asphalt modification at the material level and structural performance prediction [4,12,15].

The 2S2P1D model ("Two Springs, Two Parabolic elements, One Dashpot") was developed by [21-22] as an advancement of previous linear viscoelastic models, including the Huet–Sayegh formulation. Through the combination of these components, the 2S2P1D model (Table 2) is able to reproduce the complete viscoelastic spectrum of asphalt materials, from predominantly elastic behavior at short loading times to viscous flow at long loading durations. These effects may require additional modelling approaches. The accurate prediction of asphalt mixture behaviour requires constitutive models capable of capturing both temperature and loading frequency effects. Sauzéat et al. [53] proposed a thermo-viscoelastic framework for bituminous mixtures, emphasizing the importance of time-temperature dependency in asphalt response. Subsequently, Cochet et al. [24] experimentally validated the 2S2P1D model, demonstrating its capability to reproduce complex modulus behaviour over a broad range of testing conditions.

Generalized Kelvin–Voigt models (Table 2) remain essential tools in asphalt rheology because they provide a direct link between experimental material characterization and numerical simulation approaches [15,54]. Unlike single-value indicators such as modulus, the relaxation spectrum offers a more comprehensive description of the viscoelastic response by revealing how material modification influences fundamental mechanisms, including molecular mobility, relaxation processes, elastic energy storage, and viscous dissipation. The formulation proposed by Zoa et al. [25-27]; Zoa [28] extends the Generalized Kelvin–Voigt model by introducing 39 independent parameters, allowing a very detailed representation of the asphalt mixture's relaxation spectrum. Typically, these parameters include a glassy modulus E_∞, multiple Kelvin element stiffnesses E_i, corresponding relaxation times τ_i (or viscosities η_i) and additional coefficients that refine the spectrum and improve numerical stability.

Table 3 presents a global comparative evaluation of the main constitutive models applied to asphalt materials used in this study, highlighting their respective advantages, limitations, parameter requirements, and suitability for rheological characterization and pavement performance prediction.

3. Materials and Methods

The asphalt mixture investigated in this study is representative of materials commonly used for bituminous pavement surface courses. The material characterization and the experimental testing procedures adopted are described. The formulation test used is the one recommended in the standard NF P 98-150-1 [55], the summary of which is shown in Fig. 4.

3.1. Materials

3.1.1. Bituminous binder

The binder used was a conventional 35/50 penetration grade paving bitumen, commonly employed in the construction of high-performance asphalt pavements subjected to medium and heavy traffic conditions. The binder was selected because of its favorable balance between rutting resistance and fatigue performance under tropical climatic conditions. To improve the workability and adhesion properties of the asphalt mixture, CECABASE® 300 was incorporated into the binder at a dosage of 0.3% by weight of bitumen. A previous formulation study with several percentages of CECABASE® 300 showed that an optimal dosage with its addition of 0.3% allowed us to achieve good mechanical performance of the mixture confirmed by lab tests [56]. That's why the optimal dosage of 0.3% of CECABASE® 300 is used directly in this study. CECABASE® 300 is a chemical warm-mix asphalt additive that reduces the internal friction between binder and aggregates during mixing and compaction while enhancing aggregate coating and moisture resistance. Two formulations were therefore investigated:

- Reference mixture (0% CECABASE 300);
- Modified mixture (0.3% CECABASE 300).

The modified binder was prepared by adding the required amount of CECABASE® 300 to the heated binder under continuous mechanical stirring until a homogeneous blend was obtained before mixture production.

3.1.2. Aggregates

The mineral aggregates consisted of crushed gneiss (Fig. 5) obtained from the Logbadjeck quarry (Cameroon). The aggregates were selected because of their high mechanical strength, angularity, and suitability for wearing-course applications. Prior to mixture production, the aggregate grading was determined according to NF EN 12697-1 [57] and NF EN 12697-2 [58]. The particle size distribution was adjusted to satisfy the requirements for dense asphalt concrete intended for surface layers.

3.1.3. Asphalt mixture composition

The asphalt mixture was designed as a dense asphalt concrete for wearing-course applications. The optimized formulation consisted (Fig. 6 and 7) of:

- 35/50 penetration grade bitumen;
- Binder content: 5.1% (by total mixture mass);
- Polymer additive (PR PLAST S): 0.4%;
- Crushed gneiss aggregates (0/14 mm);
- CECABASE® 300: 0% and 0.3%.

The reference formulation was used for comparison with the modified mixture to quantify the influence of CECABASE® 300 on both mechanical performance and thermo-viscoelastic behavior.

3.2. Experimental program

A comprehensive laboratory investigation was performed to evaluate the physical, mechanical, and rheological properties of the modified asphalt mixture. Fig. 4 summarizes the experimental methodology adopted in this work. The experimental program was designed to comprehensively evaluate the material characteristics and mechanical performance of asphalt mixtures. The investigation included binder characterization, aggregate grading analysis, mixture design, mechanical performance evaluation, and thermo-viscoelastic modelling. The experimental activities comprised the following components:

- Binder characterization to evaluate the fundamental physical and rheological properties of the asphalt binder.
- Aggregate grading analysis to determine the particle size distribution and ensure compliance with mixture design requirements.
- Marshall mix design for the determination of optimum binder content and initial mixture volumetric characteristics.
- Gyratory compaction to assess compaction behavior and evaluate the evolution of mixture density under simulated field conditions.

- Water sensitivity evaluation to investigate moisture-induced degradation mechanisms.
- Indirect Tensile Strength ratio (ITSR) testing to quantify resistance to moisture damage.
- Wheel tracking (Rutting) testing to evaluate resistance to permanent deformation and rutting under repeated loading conditions.
- Stiffness modulus determination to characterize the load-bearing capacity and temperature-dependent mechanical response of asphalt mixtures.
- Fatigue characterization to assess resistance to crack initiation and propagation under cyclic loading.
- Dynamic modulus testing to determine the frequency- and temperature-dependent viscoelastic response of asphalt mixtures.
- Thermo-viscoelastic modelling to describe material behavior and establish relationships between laboratory characterization, constitutive parameters, and pavement performance prediction.

The combination of experimental characterization and advanced modelling provides a comprehensive framework for evaluating the influence of material properties and modification strategies on asphalt mixture performance.

3.2.1 Binder characterization

The physical properties of both binders (0% and 0.3% CECABASE® 300) were determined according to European standards. The following tests were performed: Flash point (NF EN ISO 2592[59]); Penetration at 25°C (NF EN 1426 [60]); Ring-and-ball softening point (NF EN 1427 [61]); Dynamic viscosity; Solubility (NF EN 12592 [62]; Rolling Thin Film Oven Test (RTFOT) (NF EN 12607-1 [63]; Retained penetration; and Softening point increase after aging. These tests were used to evaluate the influence of CECABASE® 300 on binder consistency, temperature susceptibility, and short-term aging resistance.

3.2.2 Aggregate grading

The particle size distribution of the crushed gneiss aggregates was determined using sieve analysis according to NF EN 12697-2 [58]. The grading curve was verified to ensure compliance with the specification limits for asphalt wearing-course mixtures.

3.2.3 Marshall mix design and Gyratory compaction

Marshall specimens were prepared according to NF P 98-251-2 [64] and NF EN 12697-34 [65]. The following properties were determined: Bulk density; Air void content; Compactness; Marshall stability and Marshall flow. The Marshall test was used to verify the structural integrity of the designed asphalt mixture.

In Gyratory compaction test, the asphalt mixture, designed with a richness modulus (K) of 3.5, was produced by mixing the constituent materials using a GZM-30+ laboratory mixer at production temperatures of 175°C (for reference material) and 175°C (material of 0.3% CECABASE 300). Cylindrical specimens with an average diameter of 150 mm and a minimum height of 150 mm. The compactor was operated at a rotation speed of 30 rpm, with a gyration angle of 0.82° and a vertical compaction force of 11.7 kN (Fig. 8). The compactability of the mixtures was evaluated using a French gyratory compactor in accordance with NF EN 12697-31 [66] and NF EN 12697-10 [67]. The evolution of specimen height, density, compactness and air void content was continuously monitored as a function of the number of gyrations (1 to 200 gyrations). This test was employed to assess the effect of CECABASE® 300 on workability and field compaction characteristics.

3.2.4 Water sensitivity

The asphalt mixture was prepared by mixing the constituent materials in a GZM-30+ laboratory mixer at mixing and molding temperatures of 175°C. Specimens were compacted in 80-mm diameter molds. The conditioning period between specimen preparation and testing was 7 days. Moisture susceptibility was evaluated using the Duriez test (NF EN 12697-12B, [68]). Eight cylindrical specimens were prepared for each formulation. Half of the specimens

were tested (Figure 9) under dry conditions, whereas the remaining specimens were conditioned in water before compression testing. The Duriez ratio was calculated as :

$$\frac{r}{R} = \left(\frac{R_w}{R_d}\right).100 \quad (9)$$

where : R_w is the compressive strength after water conditioning and R_d is the compressive strength under dry conditions.

3.2.5 Indirect Tensile Strength Ratio (ITSR)

The asphalt mixture was prepared by mixing the constituent materials in a GZM-30+ laboratory mixer at both mixing and molding temperatures of 175°C. Cylindrical specimens (Fig. 10) were compacted in 100-mm diameter molds. The conditioning period between specimen preparation and testing was 3 days. The moisture resistance of the modified asphalt mixture was also evaluated according to NF EN 12697-12A [68]. Indirect tensile strength was measured for dry and conditioned specimens. The ITSR was calculated using :

$$ITSR = \left(\frac{ITS_w}{ITS_d}\right).100 \quad (10)$$

where : ITS_w represents the tensile strength after conditioning and ITS_d represents the dry tensile strength.

3.2.6 Rutting test

The asphalt mixture was prepared using the GZM-30+ laboratory mixer at both mixing and compaction temperatures of 175°C. Two asphalt slabs were manufactured and conditioned for 4 days prior to testing. Slab compaction was carried out using a pneumatic roller in accordance with the relevant standard. Resistance to permanent deformation was determined using the wheel tracking test according to NF EN 12697-22 [69]. Testing was conducted at 60°C (Fig. 11). Rut depth evolution was monitored up to 30,000 loading cycles, allowing the evaluation of high-temperature rutting susceptibility.

3.2.7 Stiffness modulus

The stiffness modulus was determined using two complementary procedures.

3.2.7.1 Trapezoidal specimen test

Four trapezoidal specimens were tested according to NF EN 12697-26 Annex A [70] at 15°C and 28°C, using a sinusoidal loading frequency of 10 Hz. The asphalt mixture was produced using the GZM-30+ laboratory mixer at a mixing temperature of 175°C, resulting in an average air void content of 7.4%. Test slabs were then constructed on a full-scale (Fig. 12) pavement using the investigated asphalt mixture and compacted with a roller compactor, in accordance with EN 12697-26. Trapezoidal specimens were subsequently sawn (Fig. 13) from the slabs for complex modulus testing (EN 12697-26, [70]). Specimens whose dimensions did not comply with EN 12697-24 [71], were discarded. Two-point bending stiffness modulus tests were performed on specimens conditioned for 14 days at test temperatures of 15°C and 28°C and a loading frequency of 10 Hz.

3.2.7.2 Cylindrical specimen test

The asphalt mixture was produced using the GZM-30+ laboratory mixer at a mixing temperature of 175°C, with an average air void content of 7.4%. Test slabs were subsequently constructed on a full-scale pavement using the investigated asphalt mixture and compacted with a roller compactor, in accordance with EN 12697-26 [70]. Cylindrical cores were extracted from the slabs (Fig. 14) and tested using the indirect tensile stiffness modulus method after 14 days of conditioning. The tests were conducted at temperatures of 10°C and 15°C and a loading frequency of 10 Hz. Indirect tensile modulus measurements were performed on 29 cylindrical specimens according to NF EN 12697-26 Annex C [70]. Statistical parameters including mean value, standard deviation, minimum, and maximum values were subsequently calculated.

3.2.8 Fatigue characterization

The asphalt mixture was prepared using the GZM-30+ laboratory mixer at a mixing temperature of 175°C, resulting in an average air void content of 7.6%. Test slabs were then constructed on a full-scale pavement using the investigated asphalt mixture and compacted with a roller compactor, in accordance with NF EN 12697-26 [69]. Trapezoidal specimens (Fig. 15) were sawn from the slabs for complex modulus (NF EN 12697-26 [70]. and fatigue testing (NF EN 12697-24 [71].). Specimens that did not comply with the dimensional requirements of EN 12697-24 were discarded. Fatigue tests were carried out on specimens conditioned for 14 days at a test temperature of 10°C and a loading frequency of 25 Hz. The experimental setup consisted of a Data Physics control system, a power amplifier, and an electrodynamic shaker used to apply the prescribed loading signal to the specimen. A climatic chamber maintained a constant testing temperature throughout the experiments.

3.3 Dynamic modulus testing and TTSP

The linear viscoelastic behavior was characterized through dynamic modulus measurements. Complex modulus tests were carried out at several temperatures, and loading frequencies: 5, 10, 15, 20, 25, 30 and 35 Hz. These data constituted the experimental basis for thermo-viscoelastic modeling. The experimental data were extended over a wide frequency range using the Time–Temperature Superposition Principle (TTSP). The master curve at the reference temperature of 15°C presented herein was constructed using a thermo-rheological model based on the Arrhenius and WLF law, from experimental data measured at many temperature. In the conventional time–temperature superposition principle (TTSP) approach, the shift factors are determined from complex modulus tests performed at several temperatures. The experimental modulus–frequency curves obtained at the different temperatures are then horizontally shifted to construct an experimental master curve. The reduced frequency was calculated as :

$f_r = f \times a_T$

where : f_r is the reduced frequency, f is the loading frequency, a_T is the temperature shift factor.

The shift factors were determined using the Arrhenius and WLF equations at the reference temperature (15°C). The resulting master curve covered temperatures ranging from −10°C to 40°C.

3.4 Constitutive viscoelastic modeling

The experimental master curve was calibrated using three constitutive rheological models : Huet–Sayegh model, 2S2P1D model and Generalized Kelvin–Voigt model. A 39-parameter Generalized Kelvin–Voigt model composed of 19 Kelvin elements and one equilibrium spring was employed to reproduce the experimental master curve. The optimization was performed by minimizing the Root Mean Square Error (RMSE) between the experimental and predicted complex moduli.

$$Error = \sum \left(\left| E^* \right|_{\exp} - \left| E^* \right|_{\mathrm{mod}\,el} \right)^2 \tag{11}$$

4. Results of test investivations

These results provide a good basis for discussing how 0.3% Cecabase® 300 affects the fundamental properties of a 35/50 penetration grade bitumen before mixture production. The discussion is focused on the physical mechanisms and their implications for asphalt mixture performance rather than simply reporting numerical differences.

4.1 Effect of Cecabase®300 on the Physical Properties of the 35/50 Bitumen

Table 4 summarizes the conventional characterization of the 35/50 penetration grade bitumen before and after modification with 0.3% Cecabase® 300.

Overall, the additive induced only minor changes in the intrinsic properties of the binder while preserving compliance with the specifications for the base bitumen. The penetration at 25°C decreased slightly from 43 dmm for the unmodified binder to 41 dmm after the addition of Cecabase® 300, indicating a marginal increase in binder consistency. Although the reduction is small (approximately 5%), it suggests that the additive does not significantly soften the binder despite being designed as a warm-mix asphalt (WMA) additive. This observation is advantageous because excessive softening could reduce the rutting resistance of asphalt mixtures under high service temperatures. The ring-and-ball softening point increased from 51.4°C to 53.2°C, corresponding to an increase of approximately 1.8°C. A higher softening point generally indicates improved resistance to flow and permanent deformation at elevated temperatures. The simultaneous decrease in penetration and increase in softening point suggest a slight stiffening effect of the modified binder. Such behavior has been reported for several surfactant-based WMA additives, where the additive improves the internal organization of the bituminous phase without substantially altering the chemical composition of the binder. The relatively small magnitude of these changes also indicates that Cecabase® 300 acts primarily as a processing aid rather than as a polymeric modifier. Unlike elastomeric or plastomeric modifiers, which substantially alter the rheological behavior of bitumen, surfactant-based additives mainly improve coating and workability during mixing and compaction while maintaining the original binder classification.

4.2 Aggregate Gradation of the Logbadjeck quarry (Cameroon)

The gradation analysis serves to demonstrate that aggregate grading was controlled, ensuring that any differences in mechanical performance can be attributed to the binder formulation rather than changes in aggregate structure. The particle-size distributions of the asphalt mixtures prepared with 0% and 0.3% Cecabase® 300 are presented in Fig. 16. The aggregates were sourced from the Logbadjeck quarry (Cameroon) and characterized according to NF EN

12697-2. The gradation curves of both formulations are remarkably similar over the entire particle-size range, indicating that the same aggregate blend was used in both mixtures. Minor differences in the percentage passing at individual sieve sizes remain small and are consistent with normal experimental variability associated with aggregate batching, sieving, and sample preparation. The fine fraction (<0.25 mm) shows only limited variation between the two formulations. For example, the percentage passing the 0.063 mm sieve was 5.9% for the control mixture and 5.8% for the mixture containing Cecabase® 300. The intermediate aggregate sizes (0.5–6.3 mm), which contribute significantly to the internal aggregate skeleton, also exhibited very similar distributions. Slight variations, such as 65% versus 60% passing the 5 mm sieve or 77.9% versus 75% passing the 6.3 mm sieve, are not sufficient to modify the fundamental packing characteristics of the mixture. Consequently, both formulations can be considered to possess comparable aggregate interlock and stone-to-stone contact.

4.3 Marshall Performance of the Asphalt Mixture Containing Cecabase® 300

The Marshall properties of the asphalt mixture prepared with the modified 35/50 penetration grade bitumen are presented in Table 5. The results indicate that all measured parameters satisfy the specification requirements, confirming that the incorporation of 0.3% Cecabase® 300 does not compromise the structural integrity of the asphalt mixture. The bulk density (MV_a) of the compacted specimens reached 2.390 t/m³, reflecting effective aggregate packing and adequate compaction during specimen preparation. Although density alone does not directly characterize mechanical performance, it strongly influences the internal structure of the mixture by controlling the distribution of air voids and the aggregate skeleton. The measured value suggests that the warm-mix additive maintained sufficient workability to achieve a dense and homogeneous structure. The compactness of 93.9% falls comfortably

within the specified range of 92–96%, while the corresponding air void content of 6.1% also satisfies the required limits of 4–8%. These two parameters are closely related and indicate that the compaction process was effective without producing either an excessively dense mixture, which could increase the risk of bleeding, or an overly porous structure, which would accelerate moisture infiltration and oxidative aging. The results therefore suggest that Cecabase® 300 promotes adequate coating and lubrication of aggregate particles, facilitating compaction while preserving the volumetric characteristics required for long-term pavement durability.

The Marshall creep value of 3.15 mm lies near the middle of the specified interval (2–4 mm), indicating an appropriate balance between stiffness and flexibility. Flow values within this range imply that the mixture possesses sufficient deformation capacity to accommodate traffic-induced stresses without exhibiting excessive plastic deformation. Extremely low flow values generally indicate a brittle mixture susceptible to cracking, whereas excessively high values may reflect insufficient stiffness and an increased tendency toward rutting. The measured result therefore suggests that the modified mixture achieves a favorable compromise between these two failure mechanisms. The most remarkable result is the Marshall stability, which reached 1577 kg, nearly twice the minimum specification requirement of 800 kg. Such a high stability value demonstrates the excellent load-bearing capacity of the asphalt mixture and indicates that the aggregate skeleton and bituminous binder develop strong internal cohesion under compressive loading. This improvement can be attributed not only to the inherent properties of the 35/50 penetration grade binder but also to the enhanced aggregate coating and improved adhesion promoted by Cecabase® 300. The surfactant-based additive reduces interfacial tension between the binder and aggregate surfaces, facilitating more uniform binder distribution and stronger aggregate-binder

interaction during mixing and compaction. These mechanisms contribute to increased resistance to permanent deformation under repeated traffic loading.

From a pavement engineering perspective, the combination of high Marshall stability with moderate flow is particularly advantageous because it indicates that the mixture combines high structural strength with sufficient flexibility. This balance is desirable for wearing courses subjected to heavy traffic, where both rutting resistance and resistance to fatigue-related damage are essential for long-term performance. Overall, the Marshall test demonstrates that the asphalt mixture incorporating 0.3% Cecabase® 300 achieves excellent volumetric characteristics and outstanding mechanical stability while remaining fully compliant with specification requirements. These findings support the use of the additive as a warm-mix technology capable of improving mixture workability without compromising, and potentially enhancing, the structural performance of asphalt pavements.

4.4 Compactability of Asphalt Mixtures Evaluated Using the Gyratory Compactor

The compactability of the asphalt mixtures was evaluated using the gyratory compactor in accordance with NF EN 12697-31. Three specimens were prepared for each formulation, namely the reference mixture containing conventional 35/50 bitumen and the mixture incorporating 0.3% Cecabase® 300. The evolution of air voids and compactness with the number of gyrations is presented in Fig. 17. The two mixtures exhibited the typical densification behavior expected for asphalt mixtures subjected to gyratory compaction, characterized by a progressive reduction in specimen height and air void content as the number of gyrations increased. However, the mixture containing Cecabase® 300 consistently achieved higher densities at every compaction stage than the reference mixture. During the initial stages of compaction, both mixtures exhibited similar compactness values, with approximately 77.9% for the reference mixture and 77.5% for the modified mixture after the

first gyration. As compaction progressed, the beneficial effect of the warm-mix additive became increasingly evident. At 30 gyrations, the compactness increased from 87.44% for the control mixture to 89.40% for the modified mixture, corresponding to an improvement of approximately 2 percentage points. This trend continued throughout the test, demonstrating that the additive facilitated particle rearrangement under compaction. The greatest differences were observed at the end of the compaction process. After 200 gyrations, the compactness reached 92.57% for the reference mixture compared with 95.80% for the mixture containing Cecabase® 300, representing an increase of approximately 3.2 percentage points. Correspondingly, the final air void content decreased from 7.43% to 4.20%, corresponding to a reduction of approximately 43%.

These results indicate that the incorporation of Cecabase® 300 significantly improves the compactability of the asphalt mixture. Unlike polymer modifiers, which primarily alter the rheological properties of the binder, Cecabase® 300 functions as a surfactant that reduces interfacial tension between the binder and aggregate surfaces. The improved compactability observed throughout the entire compaction process confirms the effectiveness of the additive as a warm-mix technology and provides a mechanistic explanation for the favorable Marshall characteristics obtained for the modified mixture. Several quantitative observations strengthen the discussion (Table 6):

Similarly, the reduction in air void content becomes increasingly pronounced:

- 100 gyrations: 9.11% to 6.29% (≈31% reduction)
- 150 gyrations: 8.11% to 5.05% (≈38% reduction)
- 200 gyrations: 7.43% to 4.20% (≈43% reduction)

The progressive widening of the gap between the two compaction curves is particularly significant because it shows that the additive's influence persists throughout the densification process rather than being limited to the initial stages.

4.5 Water sensitivity of Asphalt mixtures (Duriez Test)

The Duriez test is arguably the most important durability test in this study because it evaluates the water sensitivity of asphalt mixtures, one of the primary indicators of long-term pavement performance. The control blend and the Cecabase® 300 blend were produced at 175°C. The water sensitivity of the asphalt mixtures was evaluated using the Duriez method in accordance with NF EN 12697-12 (Method B). The test compares the compressive strength of dry specimens (R_d) with that of water-conditioned specimens (R_w), and the Duriez coefficient (r/R) was used to assess the resistance of the mixtures to moisture-induced damage. The reference mixture exhibited an average initial air void content of 9.2% and an average bulk density of approximately 2.24 g/cm³, whereas the mixture containing 0.3% Cecabase® 300 showed a slightly lower air void content (8.6%) and a higher bulk density of approximately 2.28–2.30 g/cm³. These volumetric differences are consistent with the gyratory compaction results, which demonstrated improved densification in the presence of the warm-mix additive. The dry compressive strength of the reference mixture ranged from 11.99 to 12.88 MPa, while the water-conditioned specimens exhibited strengths between 6.74 and 7.57 MPa, indicating a substantial loss of mechanical resistance after moisture conditioning. Consequently, the Duriez coefficient reached only 58.02%, reflecting a significant reduction in strength due to the combined effects of water infiltration and reduced binder–aggregate adhesion (Fig. 18).

In contrast, the mixture containing 0.3% Cecabase® 300 exhibited markedly higher compressive strengths under both conditioning states. The dry specimens achieved strengths ranging from 13.60 to 15.25 MPa, whereas the conditioned specimens maintained strengths between 10.98 and 12.49 MPa. The corresponding Duriez coefficient increased to 80.37%, representing an improvement of approximately 22 percentage points relative to the reference mixture. This increase in the Duriez coefficient is highly significant because it demonstrates a

substantial enhancement in moisture resistance. The ratio increased from 0.58 to 0.80, corresponding to a relative improvement of approximately 38% in retained strength after water conditioning. Such an increase indicates that the modified mixture preserves a much larger proportion of its mechanical capacity following moisture exposure. For Duriez results, several quantitative observations strengthen the discussion (Table 7):

Several aspects stand out:

- The 22.35 percentage point increase in the Duriez coefficient is likely to be viewed as a major improvement in moisture resistance.
- The wet compressive strength increased much more than the dry strength, indicating that the additive is particularly effective in mitigating moisture-induced damage rather than simply increasing overall mixture strength.
- The Duriez ratio of 80.37% approaches or exceeds the acceptance criteria commonly specified for high-quality asphalt mixtures, while 58.02% indicates considerable susceptibility to moisture damage.

4.6 Moisture Susceptibility Evaluated by the Indirect Tensile Strength Ratio (ITSR)

I notice that, this study provide only the results for the mixture containing 0.3% Cecabase® 300. The reported ITSR value can still be interpreted with respect to commonly accepted performance criteria. The moisture susceptibility of the asphalt mixture incorporating 0.3% Cecabase® 300 was evaluated using the Indirect Tensile Strength Ratio (ITSR) test in accordance with NF EN 12697-12 (Method A) [69]. Four specimens were tested in the dry condition and four additional specimens were subjected to water conditioning before testing. The dry specimens exhibited indirect tensile strengths ranging from 2719 to 3338 kPa, with an average value of approximately 3063 kPa. After moisture conditioning, the tensile strengths ranged from 2281 to 2728 kPa, averaging approximately 2493 kPa. Although water

exposure reduced the tensile strength of the mixture, the retained mechanical capacity remained relatively high. The calculated ITSR was 81.39% (Fig. 18), indicating that more than four-fifths of the original indirect tensile strength was preserved after water conditioning. This value exceeds the 80% threshold that is commonly specified in European pavement engineering practice for asphalt mixtures subjected to moisture sensitivity evaluation. Consequently, the modified mixture can be considered to possess satisfactory resistance to moisture-induced deterioration. Overall, the ITSR value of 81.39% confirms that the asphalt mixture incorporating 0.3% Cecabase® 300 possesses satisfactory resistance to moisture damage. Together with the favorable Duriez results, these findings demonstrate that the chemical warm-mix additive enhances the durability of the asphalt mixture by preserving the mechanical integrity of the binder–aggregate system following water conditioning. The calculated average strengths are:

- Average dry ITS: ≈ 3063 kPa
- Average wet ITS: ≈ 2493 kPa
- Strength loss after conditioning: ≈ 18.6%
- ITSR: 81.39%

These values indicate that the mixture retains a high proportion of its tensile strength after conditioning, supporting good moisture resistance. An interesting aspect of this study is the consistency between the different durability indicators (Table 8):

4.7 Rutting Resistance of the Reference Asphalt Mixture

The rutting test is one of the key performance tests for a wearing course because it directly evaluates the mixture's resistance to permanent deformation under repeated loading at elevated temperature. Since rutting is a major distress mode in hot climates such as Cameroon, these results are highly relevant for the practical significance of this study. The resistance to permanent deformation of the reference asphalt mixture was evaluated using the

wheel-tracking test in accordance with NF EN 12697-22 [69]. The specimens were tested at an average temperature of 60.3°C, representative of severe service conditions for asphalt pavements exposed to high ambient temperatures and intense solar radiation. The rut depth increased progressively with the number of loading cycles, which is characteristic of asphalt mixtures subjected to repeated wheel loading. The initial rut depth was 0.86% after 30 cycles, increasing gradually to 2.33% after 30,000 cycles (Fig. 19). The corresponding reduction in specimen thickness was limited, from 100 mm to 97.67 mm, indicating that the mixture maintained its structural integrity throughout the test. The evolution of rutting was characterized by a relatively rapid increase during the early loading stages, followed by a slower rate of deformation as the number of cycles increased. Such behavior reflects the typical densification phase of asphalt mixtures, during which aggregate particles rearrange and residual air voids are reduced, followed by a more stable phase dominated by the resistance of the aggregate skeleton to plastic deformation. The initial air void content of 6% and the bulk density of 2.398 g/cm³ indicate that the specimens were compacted within an appropriate volumetric range before testing.

The final rut depth of 2.33% after 30,000 cycles demonstrates a relatively high resistance to permanent deformation. At a testing temperature exceeding 60°C, the bituminous binder experiences a considerable reduction in stiffness, making rutting resistance largely dependent on the aggregate skeleton and the quality of the binder–aggregate interaction. The limited increase in rut depth throughout the loading history suggests that the mixture developed a stable internal structure capable of effectively distributing the applied stresses. From an engineering perspective, this level of rutting indicates that the reference mixture possesses satisfactory resistance to permanent deformation under high-temperature conditions. Such performance is desirable for wearing courses subjected to heavy traffic, where excessive rutting can compromise ride quality, reduce skid resistance, and increase the risk of water

accumulation on the pavement surface. Overall, the wheel-tracking test demonstrates that the reference asphalt mixture exhibits good rutting resistance under severe loading conditions. Permanent deformation is relatively low after 30,000 cycles. The progression of rut depth is gradual (Table 9):

Several observations are noteworthy:

- The rut depth remains well below 3% after 30,000 cycles, indicating good resistance to permanent deformation under the prescribed test conditions.
- The rate of rut growth decreases with increasing cycles, suggesting that most densification occurs during the initial loading stage and that the aggregate skeleton remains stable thereafter.
- Testing at 60.3°C makes the result particularly meaningful because rutting is most critical at elevated pavement temperatures.

4.8 Stiffness Modulus of Asphalt Mixtures

The stiffness modulus results are among the most technically significant in this study because they quantify the structural response of the asphalt mixture under cyclic loading. Unlike the Marshall or Duriez tests, the stiffness modulus (NF EN 12697-26, Annex A, [70]) is a fundamental mechanistic parameter used in pavement design. It directly reflects the ability of the asphalt layer to distribute traffic loads and is closely related to rutting resistance, fatigue performance, and pavement service life. The manufacturing temperature reported in this study are 175°C for the control mixture and the Cecabase® 300 mixture. The stiffness modulus of the asphalt mixtures was determined using the trapezoidal specimen method in accordance with NF EN 12697-26 (Annex A) [70] at a loading frequency of 10 Hz. Four specimens were tested for each formulation at 15°C and 28°C to evaluate the influence of 0.3% Cecabase® 300 on the viscoelastic response of the mixtures.

4.8.1 Stiffness modulus at 15°C

At 15°C, the reference mixture exhibited stiffness modulus values ranging from 10,796 to 11,736 MPa, with an average modulus of 11,167 MPa. The mixture incorporating 0.3% Cecabase® 300 showed significantly higher values (Fig. 20), ranging from 12,563 to 14,616 MPa, with an average modulus of 13,561 MPa. The incorporation of Cecabase® 300 therefore increased the stiffness modulus by approximately 2,394 MPa, corresponding to an improvement of approximately 21.4% compared with the reference mixture. This increase indicates that the modified asphalt mixture possesses greater resistance to elastic deformation under repeated traffic loading at intermediate service temperatures. The relatively small variation between replicate specimens also indicates good repeatability of the test results, suggesting that the observed increase is systematic rather than resulting from experimental variability.

4.8.2 Stiffness modulus at 28°C

As expected for viscoelastic materials, increasing the testing temperature from 15°C to 28°C resulted in a marked reduction in stiffness for both mixtures due to the temperature-dependent behavior of the bituminous binder. The average modulus of the reference mixture decreased from 11,167 MPa at 15°C to 5,388 MPa at 28°C, representing a reduction of approximately 52%. Similarly, the modified mixture decreased from 13,561 MPa to 7,641 MPa, corresponding to a reduction of approximately 44%. Although both mixtures became more compliant at elevated temperature, the mixture containing Cecabase® 300 consistently maintained a substantially higher stiffness than the reference mixture. At 28°C, the average modulus increased from 5,388 MPa for the control mixture to 7,641 MPa for the modified mixture, corresponding to an increase of approximately 42%. The greater relative improvement observed at 28°C compared with 15°C suggests that the beneficial influence of Cecabase® 300 becomes more pronounced as temperature increases. This behavior is particularly advantageous for wearing courses operating in tropical climates, where asphalt

pavements are frequently exposed to elevated pavement temperatures that promote permanent deformation. The average values of stiffness modulus are summarized in table 10.

An important finding is that the modified mixture loses less stiffness as temperature increases. This indicates a lower sensitivity to temperature, which is particularly advantageous in regions with high pavement temperatures, such as Cameroon.

4.9 Discussion in the context of the entire study

These results from the stiffness module are consistent with the general trends observed throughout this experimental program:

- Binder characterization: Cecabase® 300 produced only minor changes in conventional binder properties.
- Gyratory compaction: the additive significantly improved mixture densification.
- Marshall test: the denser internal structure resulted in high load-bearing capacity.
- Duriez and ITSR: improved binder–aggregate interaction enhanced moisture resistance.
- Stiffness modulus: the modified mixture exhibited substantially higher structural rigidity, especially at elevated temperature.

This progression supports a coherent mechanistic interpretation: the principal benefit of Cecabase® 300 is not a major alteration of binder rheology but an improvement in mixture architecture through enhanced coating, compaction, and aggregate–binder adhesion. These microstructural improvements translate into higher stiffness, better moisture durability, and greater resistance to deformation.

4.10 Stiffness Modulus Determined by Indirect Tensile Loading

The stiffness modulus of the asphalt mixture containing 0.3% Cecabase® 300 was determined using cylindrical specimens subjected to indirect tensile loading according to NF EN 12697-26 Annex C [70]. A total of 29 specimens were tested under controlled loading conditions at two temperatures (15°C and 10°C) with a loading frequency close to 8 Hz. The specimens exhibited good manufacturing consistency, with an average bulk density of 2.34 t/m³ and an average compactness of 92.09%. The compactness values ranged from 88.17% to 96.21%, indicating satisfactory control of specimen preparation. The relatively limited standard deviation of compactness (1.98%) confirms good homogeneity among the tested specimens (Table 11).

4.10.1 Stiffness modulus at 15°C

At 15°C, the measured stiffness modulus values ranged from 7684 MPa to 13998 MPa, with an average value of: $E_{15°C}$ = 10968 MPa. The standard deviation was 1613 MPa, corresponding to a coefficient of variation of approximately 14.7%, which is consistent with the variability commonly observed in asphalt mixture stiffness measurements due to differences in specimen density, aggregate arrangement, and internal heterogeneity. The obtained modulus values indicate that the mixture possesses a relatively high structural rigidity at intermediate temperature. This behavior suggests an efficient stress transfer mechanism within the asphalt matrix, resulting from the combination of adequate aggregate packing, effective binder coating, and strong aggregate–binder interaction. The comparison between 10°C and 15°C demonstrates that the modified mixture maintains significant stiffness across different temperatures (Fig. 20). This is important for pavement applications in Cameroon, where daily and seasonal temperature variations may induce changes in mixture behavior.

4.10.2 Comparison with trapezoidal stiffness modulus results

The cylindrical specimen results are consistent with the previous trapezoidal beam test (Table 12).

The difference between both methods is only approximately 3%, demonstrating excellent agreement between independent testing configurations. This agreement significantly strengthens the reliability of the stiffness characterization because two different specimen geometries and loading modes lead to comparable results.

The measured stiffness modulus indicates that the Cecabase® 300-modified asphalt mixture can be classified as a highly rigid asphalt material suitable for structural wearing course applications. The average normalized modulus of approximately 13.2 GPa demonstrates enhanced mechanical performance, characterized by high resistance to traffic-induced deformation, efficient stress distribution within the asphalt layer, and potential reduction of tensile strain accumulation under repeated loading conditions. These stiffness characteristics are consistent with the complementary mechanical performance indicators obtained from the mixture evaluation, including:

- Marshall stability of 1577 kg;
- Duriez ratio of 80.37%;
- Indirect tensile strength ratio (ITSR) of 81.39%;
- Improved gyratory compactability.

The combined results confirm that Cecabase® 300 contributes to the development of a mechanically robust asphalt mixture with improved structural integrity and resistance to traffic-related deterioration mechanisms. The performance enhancement associated with Cecabase® 300 appears to result primarily from microstructural optimization, including improved aggregate–binder interactions and mixture densification, rather than from substantial chemical modification of the binder itself.

4.11 Fatigue Performance of the Reference Asphalt Mixture

The fatigue test is the highest-level mechanical performance test in this study. It directly assesses the long-term structural durability of the asphalt mixture under repeated traffic loading. This section presents a fatigue dataset, corresponding to 16 trapezoidal specimens tested at three strain levels (88, 105, and 137 με).

The fatigue resistance of the asphalt mixture was evaluated using the two-point bending test on trapezoidal specimens in accordance with NF EN 12697-24-Annex A [71]. The tests were performed at 10°C and a loading frequency of 25 Hz, using three controlled strain levels of 88, 105, and 137 με (Fig. 21). As expected, fatigue life decreased with increasing strain amplitude, demonstrating the characteristic strain-dependent behavior of bituminous materials. At the lowest strain level (88 με), the specimens exhibited fatigue lives ranging from 1.45×10^6 to 4.90×10^6 cycles, indicating excellent resistance to repeated loading under relatively low tensile strain. Increasing the strain level to 105 με produced a noticeable reduction in fatigue life, with failure occurring between 0.55×10^6 and 2.40×10^6 cycles. This decrease reflects the greater tensile damage accumulated during each loading cycle as the imposed deformation increased. At the highest strain level (137 με), fatigue resistance decreased further, with specimen lives varying between 0.16×10^6 and 0.61×10^6 cycles. These results illustrate the nonlinear nature of fatigue damage in asphalt mixtures, whereby relatively small increases in tensile strain produce disproportionately large reductions in service life.

The observed behavior reflects the progressive accumulation of microcracks within the bituminous matrix:

1. At low strain (88 με), damage accumulates slowly, allowing the mixture to survive several million load applications.
2. At intermediate strain (105 με), crack initiation occurs earlier and crack propagation accelerates.
3. At high strain (137 με), the rate of damage accumulation increases markedly, leading to premature failure.

These mechanisms are consistent with the higher stiffness measured at 15°C, since stiffer mixtures generally experience lower tensile strains under a given load but may exhibit greater sensitivity if the imposed strain becomes sufficiently high.

4.12 Complete discussion integrating all the tests into a single mechanistic interpretation

Several observations emerge immediately. The binder properties changed only slightly, whereas the volumetric and mechanical properties of the asphalt mixture improved markedly. This contrast strongly supports the hypothesis that the additive acts mainly through microstructural optimization rather than binder hardening (Table 13 and 14).

The gyratory compaction test demonstrated that the Cecabase® 300-modified mixture achieved a compactness of 95.8%, compared with 92.6% for the reference mixture. This improvement confirms the positive influence of the additive on mixture densification and internal structural organization. Higher compactness promotes:

- increased aggregate contact area;
- improved aggregate interlock;
- enhanced load transfer efficiency within the mixture skeleton.

As a consequence of this improved internal structure, the modified mixture exhibited significant stiffness increases, with improvements of:

- approximately 21% at 15°C;
- approximately 42% at 28°C.

These results indicate that the stiffness enhancement is primarily associated with improved mixture architecture rather than substantial changes in binder properties. The denser aggregate skeleton provides more efficient stress distribution and contributes to improved resistance against traffic-induced deformation. The gyratory compaction results demonstrated a significant reduction in air voids: 7.43%→4.20%. Lower air void content reduces: permeability; interconnected pore volume and water circulation. Consequently, Duriez ratio increased: 58.02% → 80.37%. The relationship can be summarized by $r/R = f(V_a)$.

where V_a = air void content.

Lower voids lead to greater retained strength.

Mechanistic pavement design theory establishes a direct relationship between asphalt mixture stiffness and the tensile strains developed under traffic loading. Although fatigue test results are currently available only for the reference mixture, the significant increase in stiffness observed for the Cecabase® 300-modified mixture suggests that it may experience lower tensile strain levels under equivalent in-service loading conditions. This interpretation is based on established mechanistic relationships between modulus, structural response, and fatigue damage accumulation. However, the expected improvement in fatigue performance of the modified mixture remains a hypothesis and should be confirmed through direct fatigue testing.

5. Viscoelastic modelling of the BBTD

So far, the experimental campaign presented in this paper has shown the performance of the asphalt mix. Viscoelastic modeling will improve the study because it explains the material's behavior in terms of constitutive models and allows predictions beyond the tested conditions. The results from laboratory tests on complex modulus at a temperature of 15°C trials with

bituminous material doped with 0.3% of Cécabase 300 are presented in the Table 15. The first observation is that the complex modulus increases continuously wit loading frequency. The modulus rises from 12.295 GPa at 5 Hz to 14.895 GPa at 35 Hz, corresponding to an absolute increase of: 14895 – 12295 = 2600 MPa ; or a relative increase of:

$$\frac{2600}{12295}.100 = 21.1\%$$

This increase is characteristic of linear viscoelastic asphalt mixtures and reflects the reduced time available for viscous flow at higher loading frequencies. For viscoelastic materials, the relationship between modulus and frequency is commonly expressed as: E = a + bln(f).

Using these data, the approximate regression is: E = 10361+1245.7ln(f)

with an estimated coefficient of determination: $R^2 \approx 0.97$.

The high value of R^2 indicates that the change in the modulus is mainly governed by the logarithm of the loading frequency, which agrees with the classic rheological behavior. Fig. 22 clearly shows the viscoelastic stiffening of the asphalt mix as the loading frequency increases.

The measured trend indicates that the mixture is suitable for TTSP because: the modulus changes smoothly with frequency; no abrupt discontinuities are observed; the response is characteristic of a thermorheologically simple material. The values of the complex modulus measured at a temperature of 15°C closely match the stiffness values previously obtained at the same temperature using the NF EN 12697-26 standard [69]. At 10 Hz, the measured complex modulus is: |E∗|=13411 MPa. This is very close to the average stiffness of the trapezoidal specimens measured earlier: 13561 MPa.

The difference is only: $\frac{13561-13411}{13561}.100 = 1.1\%$

This excellent agreement confirms the consistency of the experimental program and indicates that the dynamic modulus measurements are reliable for constitutive model calibration.

The observed increase in complex modulus with loading frequency demonstrates that the asphalt mixture exhibits the expected linear viscoelastic behavior, in which elastic response becomes increasingly dominant as the loading duration decreases. The relatively smooth and monotonic evolution of the modulus confirms the absence of anomalous rheological behavior and supports the application of the time–temperature superposition principle. Furthermore, the high modulus values measured at 15°C (12.295 to 14.895 GPa) indicate a dense and mechanically efficient aggregate skeleton, consistent with the high Marshall stability and stiffness modulus obtained in the earlier experimental program. These results therefore provide a robust experimental basis for constructing the master curve and calibrating advanced constitutive models such as the Huet–Sayegh, 2S2P1D, and GKV models.

5.1 Time–Temperature Superposition (TTSP), Arrhenius and WLF Shift Factor Analysis

The application of the Time–Temperature Superposition Principle (TTSP) combined with the Arrhenius and WLF equations is one of the strongest aspects of this work because it transforms the experimental dynamic modulus data measured at multiple temperature and frequency into a continuous prediction of the material behavior over a wide range of service temperatures (−10°C to 50°C). This approach provides a constitutive description of the asphalt mixture that is directly applicable in mechanistic–empirical pavement design. The Arrhenius and WLF equations were used to calculate the horizontal shift factor a_T, allowing each modulus–frequency curve to be shifted to the reference temperature of 15°C. The obtained shift factors are summarized in Fig. 23.

5.2 Viscoelastic Constitutive Modeling of Asphalt Mixtures

The complex modulus completely characterizes the linear viscoelastic response and serves as the basis for advanced constitutive models such as Huet–Sayegh, 2S2P1D, and the

Generalized Kelvin–Voigt Model (GKVM). The mathematical formulations of these models were developed and presented in the literature review section. The parameters of the three constitutive models are identified by minimizing the discrepancy between the experimental complex modulus and the model predictions. Let Ei^{exp} be the measured modulus, and $Ei^{mod(\theta)}$, the modulus predicted by a model with parameter vector θ.

$(e.g., \theta = [E_0, E_\infty, k, h, \tau]$ for the Huet-Sayed model).

The optimization problem is formulated as : $\theta^* = \arg\min J(\theta)$ where the objective function is the least-quares criterion :

$$J(\theta) = \sum_{i=1}^{N} \left[E_i^{\exp} - E_i^{\mathrm{mod}}(\theta) \right]^2 \quad (12)$$

5.2.1 Determination of the Huet–Sayegh parameters

The experimental master curve obtained using the Time–Temperature Superposition Principle (TTSP), and the Arrhenius shift function was calibrated using the Huet–Sayegh viscoelastic model. The optimized parameters are summarized in Table 16. These parameters provide valuable information on the rheological behavior of the asphalt mixture over the entire frequency spectrum represented by the master curve (Fig. 24).

5.2.1.1 Interpretation of the Glassy Modulus (E_∞)

The optimized glassy modulus is E_∞ = 25.5GPa. This parameter represents the instantaneous elastic stiffness of the asphalt mixture when the loading duration approaches zero (very high reduced frequencies or very low temperatures). Several observations can be drawn.

First, the value of 25.5 GPa is considerably higher than the experimentally measured modulus at 15°C (approximately 12 to 15 GPa), indicating that the experimental frequency range (5–35 Hz) did not reach the glassy domain of the material. Consequently, the optimization extrapolates the asymptotic behavior of the master curve toward the high-frequency limit. Second, this value falls within the typical range reported for dense asphalt mixtures

(approximately 20 to 35 GPa), suggesting that the calibrated parameter is physically realistic. It reflects the stiffness of the mineral aggregate skeleton when the contribution of viscous deformation becomes negligible. From a pavement engineering perspective, a high E_∞ indicates that the mixture can effectively resist very rapid traffic loads and low-temperature loading conditions, thereby limiting instantaneous elastic strains.

5.2.1.2 Interpretation of the Static Modulus (E_0)

The optimization yields E_0 =585MPa. The static modulus represents the equilibrium stiffness reached after complete stress relaxation (very low reduced frequencies or very high temperatures). The ratio between the glassy and equilibrium moduli is

$$\frac{E_\infty}{E_0} = \frac{25500}{585} = 43.6 \tag{13}$$

This large ratio indicates a broad viscoelastic spectrum and confirms that the asphalt mixture undergoes substantial stiffness changes with loading time and temperature. Such behavior is characteristic of bituminous materials, which transition from an almost elastic response at low temperatures to a much softer response under prolonged loading or elevated temperatures. The relatively low value of E_0 is consistent with the master curve predictions at high temperatures (40 – 50°C), where the modulus decreases to approximately 0.6 to 6 GPa. It therefore confirms that the model captures the long-term relaxation behavior realistically.

5.2.1.3 Interpretation of the Fractional Exponent k

The first fractional exponent is k = 0.215. The exponent k governs the low-frequency portion of the master curve and reflects the delayed elastic response of the material. Its relatively low value indicates a gradual rather than abrupt transition from equilibrium behavior to the viscoelastic transition region. This suggests that the mixture relaxes progressively, without sudden changes in stiffness as the loading time increases. Values between 0.15 and 0.30 are commonly reported for dense asphalt mixtures; therefore, the obtained value is fully consistent with published literature.

5.2.1.4 Interpretation of the Fractional Exponent h

The second fractional exponent is h = 0.540. Unlike k, the parameter h controls the high-frequency portion of the master curve. The optimized value satisfies the theoretical requirement $0 < k < h < 1$, which guarantees the mathematical stability and physical consistency of the Huet–Sayegh model. Because h is significantly larger than k, the model predicts a steeper modulus increase in the high-frequency region than in the low-frequency region. This behavior agrees well with the experimental master curve, where the modulus rises rapidly toward the glassy plateau as the reduced frequency increases.

5.2.1.5 Interpretation of the Weighting Parameter (δ)

The optimized weighting coefficient is $\delta = 2.80$. This parameter controls the relative influence of the two fractional (parabolic) elements. A value greater than unity indicates that the first fractional element contributes more strongly to the overall relaxation process than the second. Consequently, the master curve exhibits a relatively smooth transition between the glassy and equilibrium regions rather than a sharp inflection. The resulting curve is continuous and free from oscillations, demonstrating that the optimization converged toward a physically meaningful solution.

5.2.1.6 Interpretation of the Characteristic Time (τ)

The characteristic relaxation time is $\tau = 2.40$ s. This parameter determines the horizontal position of the master curve along the reduced-frequency axis. A relaxation time of approximately 2.4 s indicates that the material requires several seconds to dissipate internal stresses fully. Consequently:

- loading events shorter than approximately 2 s produce predominantly elastic behavior;
- loading events longer than approximately 2 s allow significant viscous relaxation.

This value is compatible with the observed behavior of dense asphalt mixtures subjected to normal traffic loading.

5.2.2 Determination of the 2S2P1D parameters

The experimental master curve obtained using the Time–Temperature Superposition Principle (TTSP) and the Arrhenius shift law was calibrated with the 2 Springs, 2 Parabolic Elements, 1 Dashpot (2S2P1D) model. The optimized parameters are presented in Table 16. The optimized parameters characterize the rheological response of the asphalt mixture over the entire reduced-frequency domain represented by the master curve (Fig. 24).

5.2.2.1 Interpretation of the Glassy Modulus (E_∞)

The optimized glassy modulus is E_∞ = 25450MPa. This value represents the maximum elastic stiffness reached under extremely rapid loading or very low temperatures. It is remarkably close to the value obtained with the Huet–Sayegh model ; E_∞ = 25500MPa representing a difference of only 0.20%.

Such an agreement demonstrates that both constitutive models converge toward the same physical limit of the material and confirms the reliability of the optimization process. Moreover, the predicted glassy modulus is consistent with the master curve, which approaches approximately 25 GPa at the highest reduced frequencies. This confirms that the aggregate skeleton dominates the mechanical response under quasi-instantaneous loading conditions.

5.2.2.2 Interpretation of the Equilibrium Modulus (E_0)

The equilibrium modulus obtained after optimization is E_0 = 585 MPa, which is exactly identical to the value obtained using the Huet–Sayegh model. This modulus corresponds to the long-term equilibrium state after complete stress relaxation. It governs the behavior at:

- high temperatures,
- very low reduced frequencies,
- long loading durations.

The ratio between the glassy and equilibrium moduli is

$$\frac{E_\infty}{E_0} = \frac{25450}{585} = 43.5 \quad (14)$$

This high ratio confirms the broad viscoelastic spectrum of the BBTME mixture and its strong sensitivity to temperature and loading time.

5.2.2.3 Interpretation of the Fractional Exponent k

The optimized parameter is k = 0.20. This exponent controls the low-frequency branch of the master curve and reflects the delayed elastic response. Its relatively small value indicates that the stiffness decreases progressively as the reduced frequency decreases, producing a smooth relaxation process rather than an abrupt transition. The obtained value is very close to that obtained with the Huet–Sayegh model (k = 0.215), indicating that both models describe the low-frequency behavior in a very similar manner.

5.2.2.4 Interpretation of the Fractional Exponent h

The optimized value is h = 0.30. Unlike k, this parameter controls the intermediate and high-frequency transition of the master curve. The optimized value satisfies the theoretical requirement $0 < k < h < 1$, ensuring the mathematical consistency of the model. Compared with the Huet–Sayegh model (h = 0.54), the lower value obtained here indicates a more gradual transition toward the glassy region. Consequently, the 2S2P1D model predicts a smoother evolution of stiffness over the transition zone.

5.2.2.5 Interpretation of the Weighting Parameter (δ)

The optimized weighting parameter is $\delta = 1.55$. This coefficient determines the relative influence of the two parabolic elements on the viscoelastic response. Compared with the Huet–Sayegh model $\delta = 2.80$. The lower value suggests that the contributions of the two fractional elements are more balanced in the 2S2P1D representation. As a result, the transition between the glassy and equilibrium regions is less pronounced, producing a smoother sigmoidal master curve.

5.2.2.6 Interpretation of the Dashpot Parameter (β)

One of the main advantages of the 2S2P1D model over the Huet–Sayegh model is the explicit introduction of the Newtonian dashpot coefficient, $\beta = 27.85$. This parameter controls the irreversible viscous flow that develops during long loading times. A relatively high value of β indicates a strong resistance to viscous deformation, meaning that the mixture exhibits limited permanent flow under sustained loading. This interpretation is fully consistent with the excellent rutting resistance observed experimentally, where the rut depth remained below 2.5% after 30,000 loading cycles. Thus, the calibrated dashpot parameter provides a rheological explanation for the favorable permanent deformation performance measured in the laboratory.

5.2.2.7 Interpretation of the Characteristic Time (τ)

The optimized characteristic relaxation time is $\tau = 0.80$ s. Compared with the Huet–Sayegh model, $\tau = 2.40$ s .The relaxation time predicted by the 2S2P1D model is approximately three times smaller. This indicates that the 2S2P1D model predicts a faster stress relaxation process. Under repeated traffic loading, the material is therefore expected to dissipate internal stresses more rapidly, which may contribute to reduced stress accumulation and improved fatigue performance. The smaller relaxation time also shifts the transition region of the master curve toward higher reduced frequencies, improving the representation of the experimental data in the intermediate frequency domain.

5.2.3 Determination of the Generalized Kelvin–Voigt parameters

To further characterize the viscoelastic behavior of the BBTD asphalt mixture, the experimental master curve obtained from the Time–Temperature Superposition Principle (TTSP) was calibrated using the Generalized Kelvin–Voigt Model (GKVM) proposed by Zoa et al. [25-27] and Zoa [28]. Unlike the Huet–Sayegh and 2S2P1D models, which describe the relaxation spectrum using a limited number of fractional parameters, the GKVM represents the material by a discrete spectrum of 19 Kelvin–Voigt elements connected in parallel with an

equilibrium spring. The model therefore contains 39 parameters: one equilibrium spring (k_0); nineteen elastic springs (k_j) and nineteen dashpots (h_j).

with $\tau_j = \frac{h_j}{k_j}$, representing the characteristic retardation time of each Kelvin element.

This formulation enables the model to approximate the continuous viscoelastic spectrum of asphalt mixtures with high precision over several decades of reduced frequency (Fig. 24).

5.2.3.1 Interpretation of the equilibrium spring k_0

The optimization yields k_0 = 25 000 MPa. This value represents the instantaneous elastic stiffness of the material and corresponds approximately to the glassy modulus. Remarkably, $k_0 \approx E_\infty$ = 25.5 GPa, obtained previously with both Huet–Sayegh and 2S2P1D. The difference is less than 2 %. This excellent agreement demonstrates that the three constitutive models converge toward the same elastic limit, confirming the robustness of the experimental master curve and the optimization procedure. Table 17 presents the optimized parameters of GKVM.

5.2.3.2 Interpretation of the relaxation spectrum

The most distinctive feature of the GKVM is the distribution of the 19 Kelvin elements over an extremely broad retardation-time interval. The retardation times extend from 4.2×10^{-9} s to 4.2×10^{9} s, covering approximately18 logarithmic decades. Such a broad distribution indicates that the asphalt mixture contains relaxation mechanisms acting over an exceptionally wide range of loading durations.

Very short retardation times

Elements : 1 to 4 possess $\tau < 10^{-5}$ s. These elements govern : impact loading ; high reduced frequencies and low temperatures. They mainly represent the elastic contribution of the aggregate skeleton.

Intermediate retardation times

Elements 5 to 12 have $10^{-4} < \tau < 10^{3}$ s. These elements dominate : normal traffic loading, service temperatures, fatigue loading. They reproduce the transition zone observed experimentally in the master curve.

Very long retardation times

Elements 13 to 19 possess $\tau > 10^{3}$ s. These elements mainly control : long-term creep, permanent deformation, viscous relaxation, high-temperature response.

5.2.3.3 Evolution of Kelvin Stiffnesses

The optimized spring stiffnesses decrease continuously. Largest spring $k_1 = 1.08 \times 10^{6}$ MPa and Smallest spring k_{19} give 5 MPa. The reduction exceeds five orders of magnitude. This gradual decrease demonstrates that each Kelvin element contributes to a specific frequency interval. Unlike fractional models, the GKVM explicitly separates all relaxation mechanisms.

5.2.3.4 Evolution of dashpot viscosities

The optimized viscosities increase progressively from 4.5×10^{-3} MPa.s to 2.1×10^{10} MPa.s. This enormous range reflects the coexistence of : rapid relaxation processes, delayed elastic recovery, long-term viscous flow. Consequently, the model reproduces almost perfectly the complete viscoelastic spectrum. The three constitutive models were calibrated using the same experimental master curve obtained from TTSP. Consequently, all three models satisfy the same limiting conditions. At high reduced frequencies, all curves converge toward approximately 25 GPa. At low reduced frequencies, the predicted modulus approaches approximately 0.6 GPa. The principal differences occur in the transition region.

The comparison of the three constitutive models demonstrates that they are complementary rather than competing approaches to describing the same experimental rheological behavior. The Huet–Sayegh model is attractive because of its simplicity and physically meaningful fractional parameters, whereas the 2S2P1D model provides an improved representation of viscous flow through the introduction of a Newtonian dashpot. The GKVM achieves the

highest level of fidelity by discretizing the relaxation spectrum into nineteen Kelvin elements, allowing it to closely reproduce the experimentally determined master curve over the full reduced-frequency domain. For practical pavement design, the Huet–Sayegh and 2S2P1D models provide efficient and sufficiently accurate constitutive descriptions. However, for high-fidelity numerical modeling, inverse analysis, and detailed viscoelastic simulations, the GKVM offers the greatest predictive capability due to its superior flexibility and its ability to capture the full complexity of the asphalt mixture's relaxation mechanisms.

5.3 Statistical evaluation of the experimental master curve and constitutive model predictions

The accuracy of the constitutive models was evaluated by comparing the predicted master curves with the reference experimental master curve obtained through the Time–Temperature Superposition Principle (TTSP). The statistical indicators used were:

- Relative error (%), describing the global deviation between predicted and experimental values.
- Root Mean Square Error (RMSE), expressed in MPa, quantifying the absolute difference between predicted and measured complex modulus values.

The obtained results at the reference temperature of 15°C are summarized in Table 18. The statistical results demonstrate that all three constitutive models are capable of accurately reproducing the experimentally obtained master curve. The relative errors remain below 0.5% for all cases, confirming the validity of the calibration procedure and the appropriateness of these models for describing the linear viscoelastic behavior of the investigated BBTD asphalt mixture. The very low errors indicate that the optimized parameters successfully capture the fundamental rheological characteristics of the material: the glassy modulus region; the viscoelastic transition zone and the equilibrium modulus region. However, differences appear

in the level of precision achieved by each model, which is directly related to the mathematical flexibility of each formulation.

The ranking based on RMSE is: GKVM < 2S2P1D < Huet-Sayegh.

He indicating that the GKVM provides the closest approximation to the experimental master curve (Fig. 24). All models converge toward the same limiting values: $E_0 \approx 585$ MPa and $E\infty \approx 25500$ MPa (Fig. 24). The differences are concentrated in the transition region. Table 19 presents the model ranking and selection for Engineering applications.

The statistical analysis confirms that the optimized Huet–Sayegh, 2S2P1D, and GKVM formulations successfully reproduce the experimental master curve of the BBTD asphalt mixture. Among the investigated models, the GKVM provides the highest level of agreement with the experimental data, with the lowest RMSE (121.22 MPa) and error (0.05%). This superior performance results from its ability to represent the material response through a wide discrete relaxation spectrum. The 2S2P1D model ranks second, benefiting from the additional viscous dashpot element that improves the description of long-term behavior. The Huet–Sayegh model, despite having the highest RMSE, remains highly accurate and offers the best compromise between simplicity and physical interpretation. Therefore, the choice of constitutive model should depend on the objective of the analysis: the Huet–Sayegh model is recommended for conventional pavement design applications, the 2S2P1D model for analyses requiring realistic viscous behavior, and the GKVM for advanced predictive simulations requiring maximum reproduction accuracy of the experimental viscoelastic spectrum.

5.4 Proposition of Technical Standard of the New material tested « BBTD» (High-Modulus Tropical Asphalt Concretes)

The French design method uses the following fatigue criterion (guide of pavement, AFNOR. NF P98-086 [75]) :

$$\varepsilon_t = \varepsilon_6 \cdot \left(\frac{NE}{10^6}\right)^b \sqrt{\frac{E_{(10°C-10Hz)}}{E_{(T_{ref}-f_r)}}} . k_c . k_r . k_s \qquad (15)$$

where NE - fatigue life, number of equivalent load (13 t) applications to fatigue cracking of asphalt layer ; ε_6 – (10°C, 25 Hz) tensile strain, at which fatigue failure of a sample of asphalt mix occurs upon 10^6 loading cycles (at 50% probability) under the following test conditions: bending of beam specimen at 10°C, at frequency of 25 Hz; ε_t – tensile strain, at which fatigue failure of a base the bituminous layer; b– the slope of the fatigue line of asphalt mix; E(10°C) – the modulus of the bituminous material at 10°C; $E(\theta_{eq})$ – the modulus of the bituminous material at equivalent design temperature; k_r – factor to account for variability of test results and calculation risk; S_N – The dispersion of the results of the fatigue tests on the asphalt concrete described by the standard deviation; S_h – the thickness of the layers made of bituminous binders is variable, with a dispersion that is expressed by the standard deviation; ks – factor to account for errors in preparation of underneath asphalt layers; and k_c – factor to account for the type of asphalt mix.

The calibration coefficient (or shift factor), k_c, corrects the discrepancy between the predictions of the calculation method and the observation of the behavior of experimental pavements. Amongst other things, this coefficient implicitly integrates the effect of the discrepancies between the mechanical loadings to which the material is exposed in situ and the loadings reproduced in the laboratory tests (e.g., a state of biaxial stress with rotation of the stresses produced by a rolling load, instead of the state of uniaxial stress with a constant direction of strain in the tests on trapezoidal specimens, distinct loading kinetics of the sinusoidal curve imposed in the laboratory, periods of rest between load from traffic, etc.).

The use of the values of k_c cannot be dissociated from the calculation model proposed in this method and from the bending fatigue test protocol for trapezoidal specimens in EN 12-697-24 [71]. Unlike for roads, the high variability of airfield traffic in terms of loads leads to the adoption of a parameter k_c determined by the aggressiveness of the traffic. The evolution law considers the following points: for road traffic, the loads at the wheel is 3.25 t. Therefore, in order to guarantee the continuity of the rational method between these two fields of application, a calibration coefficient recommended is, k_c = 1.1. Tables 20 and 21 present Technical Standard of the New material tested « BBTD ». The TTSP results demonstrate the characteristic viscoelastic response of bituminous materials, with modulus increasing as loading frequency increases and decreasing as temperature rises. The smooth and continuous variation of stiffness confirms the successful application of the Time–Temperature Superposition Principle and indicates that the material behaves as a thermorheologically simple system within the tested range. The material exhibits very high stiffness at low temperatures, implying excellent load-bearing capacity but an increased potential for thermal cracking, whereas the marked reduction in modulus above approximately 35–40°C highlights greater susceptibility to permanent deformation and rutting under slow-moving or heavy traffic. Overall, these results are suitable for developing a dynamic modulus master curve and provide reliable rheological parameters for mechanistic pavement design, pavement performance prediction, and long-term structural evaluation.

6. Conclusion

This study provided a comprehensive investigation of the mechanical performance and thermo-viscoelastic behavior of a 35/50 penetration grade bitumen asphalt mixture modified with 0.3% CECABASE 300 for wearing course applications. The research combined an extensive experimental characterization program with advanced constitutive modeling approaches to establish the relationship between laboratory performance indicators and the

long-term rheological response of the material. The binder characterization results showed that the incorporation of CECABASE 300 slightly modified the rheological characteristics of the base bitumen by increasing the softening point and improving its high-temperature consistency while maintaining the required penetration grade. The additive did not adversely affect binder durability, as demonstrated by the aging resistance results, confirming its compatibility with the bituminous matrix. The complex modulus investigation confirmed the thermo-dependent and frequency-dependent nature of the modified asphalt mixture. The application of the time–temperature superposition principle combined with the Arrhenius and WLF shift function allowed the construction of a reliable master curve over a wide range of reduced frequencies at the reference temperature of 15 °C. The resulting master curve successfully described the transition from the glassy behavior at high frequencies and low temperatures to the softer viscoelastic response associated with low frequencies and elevated temperatures. Three constitutive models were calibrated and evaluated to reproduce the experimental master curve: the Huet–Sayegh model, the 2S2P1D model, and the 39-parameter Generalized Kelvin–Voigt model. All models provided excellent agreement with the experimental data, with prediction errors lower than 0.5%, demonstrating the reliability of the calibration methodology. The Huet–Sayegh model offered an accurate and simplified representation with a limited number of parameters, making it suitable for conventional pavement analysis. The 2S2P1D model provided improved prediction capability by incorporating additional viscous effects through its dashpot component, resulting in a better description of long-term relaxation behavior. The Generalized Kelvin–Voigt model achieved the highest accuracy, with the lowest RMSE value, due to its ability to represent a wide relaxation spectrum through multiple Kelvin elements.

Overall, the results demonstrate that CECABASE 300 is an effective additive for improving the mechanical performance and thermo-viscoelastic characteristics of asphalt mixtures. The

combination of experimental evaluation and advanced constitutive modeling provides a comprehensive methodology for assessing modified asphalt materials and predicting their behavior under complex traffic and environmental loading conditions. The findings contribute to the development of more durable pavement materials and provide validated constitutive models that can be implemented in advanced mechanistic pavement design and numerical simulation frameworks. Our next research will focus on field validation, long-term behavior with aging, and evaluating the modified mix under real traffic and weather conditions. The accuracy of each model was evaluated through statistical indicators to identify the most appropriate formulation for representing the thermo-viscoelastic behavior of the modified asphalt mixture. The originality of this work lies in combining comprehensive laboratory performance evaluation with advanced constitutive modeling to establish a complete understanding of the mechanical and rheological behavior of a CECABASE 300-modified asphalt mixture. The complete technical standart for the new bituminous material developed for pavement wearing courses has been presented. The results provide new insights into the potential use of this additive for high-performance pavement wearing courses and contribute to the development of more accurate predictive models for asphalt pavement design.

7. Recommendations and future perspectives

Based on the experimental results and thermo-viscoelastic modeling analysis, several recommendations are proposed for the development and use of asphalt mixes modified with CECABASE 300.

First, the incorporation of CECABASE 300 at an optimized dosage of 0.3% is recommended for improving the overall performance of asphalt wearing courses. The additive demonstrated positive effects on binder rheological properties, mixture compactability, moisture resistance, stiffness characteristics, and resistance to permanent deformation. Therefore, its use can be

considered as a promising approach for producing more durable asphalt mixtures, particularly for pavements subjected to high traffic levels and severe environmental conditions.

Second, the implementation of CECABASE 300 in asphalt production should be accompanied by an optimization of the mixing and compaction conditions. Although the additive improves workability and mixture compactability, further investigations are recommended to determine the optimal production temperatures, mixing duration, and storage conditions in order to maximize its benefits while minimizing energy consumption and potential aging effects during manufacturing.

Third, the laboratory findings should be validated through full-scale pavement trials and long-term field monitoring. The mechanical improvements observed under controlled laboratory conditions should be correlated with in-service pavement performance, including rutting evolution, fatigue cracking development, moisture damage, and seasonal variations in stiffness. Such validation would provide stronger evidence for large-scale implementation of the modified mixture.

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Data Availability Statement

The data and material used to support the findings of this study are included within the article.

Conflicts of interest

The authors confirm that there are no conflicts of interest associated with this publication.

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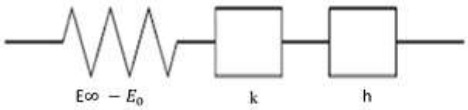
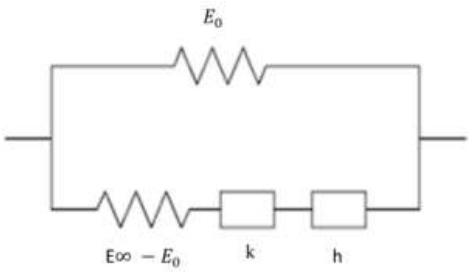
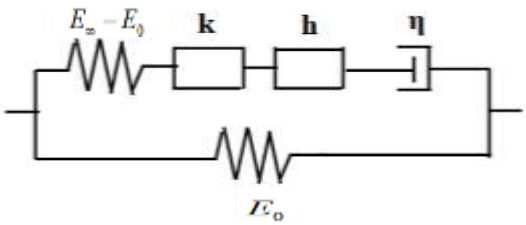
List of Tables

Table 1. Comparison between WLF and Arrhenius Models

Characteristic	WLF Model	Arrhenius Model
Origin	Polymer relaxation theory	Chemical reaction kinetics
Temperature range	Near transition region	Broad temperature ranges
Parameters	C_1, C_2	Activation energy
Physical basis	Free volume theory	Molecular activation
Asphalt application	Moderate temperatures	Wider ranges

Table 2. Viscoelastic and constitutives elements of the models used in this study

Model	Constitutives elements of the model and Complex modulus
Huet	 $E^*(\omega)=\dfrac{E_\infty}{1+\delta(i\omega\tau)^{-k}+(i\omega\tau)^{-h}}$
Huet-Sayegh	 $E^*(\omega)=E_0+\dfrac{E_\infty-E_0}{1+\delta(i\omega\tau)^{-k}+(i\omega\tau)^{-h}}$
2S2P1D	 $E^*(i\omega\tau)=E_0+\dfrac{E_\infty-E_0}{1+\delta(i\omega\tau)^{-k}+(i\omega\tau)^{-h}+(i\omega\beta\tau)^{-1}}$

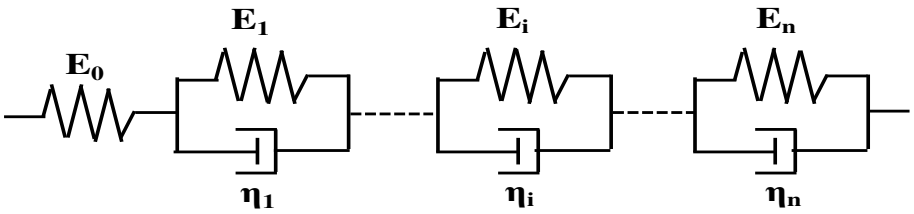
Generalized Kelvin Voigt Model	 $$E^{*KVG}(j\omega,T)=\left(\frac{1}{E_0}+\sum_{i=1}^{i=N}\frac{1}{E_i+j\omega_k\eta_i(T)}\right)^{-1}$$

Table 3. Global comparative evaluation of constitutive models used in this study

Model	Parameters	Asphalt specificity	Master curve prediction	FEM compatibility	Calibration complexity
Generalized Kelvin–Voigt	Medium–high	Medium	Excellent	Excellent	Medium
Huet–Sayegh	Low–medium	High	Excellent	Moderate	Medium
2S2P1D	Medium	Very high	Excellent	Good	High
Fractional models	Low–medium	Medium	Excellent	Moderate	High

Table 4. Results of 35/50 penetration grade bitumen with 0% and 0.3% CECABASE 300 dosage

PUR BITUMEN 35/50 WITH CECABASE 300				
Laboratory test	Standards	Specification of Standard	Values at 0.3%	Values at 0%
Penetrability of binder at 25 ºC	NF EN 1426 [60]	35-50	41	43
Ring ball softening point (TBA)	NF EN 1427 [61]	50 at 58 ºC	53.2 ºC	51.4 ºC
TBA increase	NF EN 12593 [74]	≤8 ºC		-8 ºC
Viscosity at 135 ºC	NF EN 12595 [72]			541 mm^2/s
Viscosity at 60 ºC	NF EN 12596 [73]			340 Pa. s
Solubility (m/m)	NF EN 12592 [62]		99.9%m	98.8%m
Flash point	NF EN ISO 2592 [59]		328 ºC	334 ºC
Point variation	NF EN 12607-1 [63]	≤ 0,5%	0.23%m	0.43%m
Penetrability at 25 º C in 1/10 mm				30 mm/10

PUR BITUMEN 35/50 WITH CECABASE 300			
Remaining permeability after RTFOT	≥ 53%	65%	70%
TBA remaining after RTFOT	≥ 52%		56°C

Table 5. Marshall test results performed on the modified asphalt mixture

Designations		Standards	Values	Specifications
	MVa		2.390 T/m^3	
	Compactness		93.9%	92 à 96%
Marshall	% Void	NF P 98 251 - 2	6.1%	4% à 8%
	Creep		3.15	2 à 4
	Stability		1577 kg	800 kg

Table 6. Several quantitative observations strengthen the discussion

Number of gyrations	0% Cecabase	0.3% Cecabase	Improvement
30	87.44%	89.40%	+1.96%
50	89.00%	91.37%	+2.37%
100	90.89%	93.71%	+2.82%
150	91.89%	94.95%	+3.06%
200	92.57%	95.80%	+3.23%

Table 7. Several quantitative observations strengthen the discussion for Duriez results

Parameter	0% Cecabase	0.3% Cecabase	Improvement
Dry compressive strength	12.47 MPa	14.76 MPa	+18%
Wet compressive strength	7.24 MPa	11.86 MPa	+64%
Duriez ratio (r/R)	58.02%	80.37%	+22.35 percentage points

Table 8. The consistency between the different durability indicators

Test	Indicator	Result
Gyratory compaction	Final compactness	95.8%
Duriez	r/R	80.37%
ITSR	ITS ratio	81.39%

Table 9. Gradual progression of rut depth

Loading cycles	Rut depth (%)
30	0.86
100	1.17
300	1.43
1,000	1.59
3,000	1.81

10,000	2.03
30,000	2.33

Table 10. Summarized the average values of stiffness modulus

Temperature	0% Cecabase (MPa)	0.3% Cecabase (MPa)	Increase
15°C	11166.5	13560.8	**+21.4%**
28°C	5388.0	7641.0	**+41.8%**

Table 11. The temperature sensitivity

Mixture	15°C (MPa)	28°C (MPa)	Reduction
0% Cecabase	11166.5	5388.0	**−51.8%**
0.3% Cecabase	13560.8	7641.0	**−43.7%**

Table 12. Results of stiffness Modulus determined by Indirect Tensile Loading on 29 specimens

N° of test	N° of sample	MVA (t/m^3)	Loading time (ms)	Frequency of loading (Hz)	microstrains (µm)	E (MPa) at 15°C	E (MPa) at 10°C	E (MPa) at 15°C-10Hz	MVRe (t/m^3)	Compactness (%)
1	3	2,318	123	8,13	7	9745	13030	11388	2,536	91,40
2	7	2,351	125	8,00	7	13083	18012	15547	2,536	92,71
3	10	2,383	124	8,06	7	13998	19016	16507	2,536	93,97
4	17	2,343	125	8,00	7	11518	16865	14192	2,536	92,39
5	20	2,297	124	8,06	7	12373	18324	15348	2,536	90,58
6	20	2,271	124	8,06	7	12094	15689	13892	2,536	89,55
7	28	2,35	124	8,06	7	11312	15775	13544	2,536	92,67
8	2	2,287	123	8,13	7	11150	14544	12847	2,536	90,18
9	5	2,396	123	8,13	6,9	12866	18437	15651	2,536	94,48
10	8	2,305	123	8,13	7	11242	14882	13062	2,536	90,89
11	11	2,393	124	8,06	7	12197	15776	13986	2,536	94,36
12	21	2,339	125	8,00	7,1	11864	16719	14292	2,536	92,23
13	1	2,299	124	8,06	7	9638	13641	11640	2,536	90,65
14	29	2,319	124	8,06	7	9934	14153	12043	2,536	91,44
15	26	2,281	124	8,06	7,1	10098	13715	11907	2,536	89,94
16	25	2,34	124	8,06	7	9793	14110	11951	2,536	92,27
17	22	2,258	123	8,13	7	11384	15158	13271	2,536	89,04
18	9	2,297	124	8,06	7	11006	14434	12720	2,536	90,58
19	18	2,361	124	8,06	7	7802	11195	9499	2,536	93,10
20	16	2,352	125	8,00	7,1	12005	15942	13973	2,536	92,74
21	15	2,328	123	8,13	7	9705	15453	12579	2,536	91,80
22	12	2,421	124	8,06	7	10775	16203	13489	2,536	95,47
23	4	2,236	124	8,06	7	7684	11028	9356	2,536	88,17
24	24	2,334	124	8,06	7	8541	12249	10395	2,536	92,03
25	27	2,291	124	8,06	7	8534	12826	10680	2,536	90,34
26	30	2,375	124	8,06	7	10594	15397	12995	2,536	93,65
27	6	2,42	123	8,13	6,9	12939	19035	15987	2,536	95,43

28	19	2,343	124	8,06	7	11414	15408	13411	2,536	92,39
29	14	2,44	124	8,06	6,9	12794	18650	15722	2,536	96,21
	Average	2,34	123,90	8,07	7	**10968**	**15368**	**13168**	2,536	92,09
	Max value	2,44	125	8,13	7,1	13998	19035	16507	2,536	96,21
	Min value	2,24	123	8	6,9	7684	11028	9356	2,536	88,17
	SD	0,050	0,618	0,040	0,046	1613	2209,7	1881	9E-16	1,98

Table 13. Cylindrical specimen results vs trapezoidal beam test

Test method	Temperature	Frequency	Average modulus
Trapezoidal specimen	15°C	10 Hz	13561 MPa
Cylindrical specimen	15°C–10 Hz	≈10 Hz equivalent	13168 MPa

Table 14. Overall influence of Cecabase® 300

Property	Reference	Modified	Variation
Penetration (0.1 mm)	43	41	−4.7%
Softening point (°C)	51.4	53.2	+3.5%
Mass loss after RTFOT (%)	0.43	0.23	−46.5%
Compactness after 200 gyrations (%)	92.57	95.80	+3.5%
Air voids after 200 gyrations (%)	7.43	4.20	−43.5%
Marshall stability (kg)	—	1577	—
Duriez ratio (%)	58.02	80.37	+38.5%
ITSR (%)	—	81.39	—
Stiffness modulus (15°C beam, MPa)	11167	13561	+21.4%
Stiffness modulus (28°C beam, MPa)	5388	7641	+41.8%
Stiffness modulus (5°C–10 Hz, **cylindrical specimen, MPa)**	-	13168	-

Table 15. Frequency vs Complex modulus et 15°C

F (Hz)	E (MPa) Test
5	12295
10	13411
15	13757
20	14031
25	14114
30	14678
35	14895

Table 16. Optimized Huet–Sayegh and 2S2P1D parameters at temperature of 15°C

Model	δ	k	h	E_∞ (MPa)	E_0 (MPa)	β	τ (s)
Huet–Sayegh	2.80	0.215	0.540	25500	585	-	2.40
2S2P1D	1.55	0.20	0.30	25450	585	27.85	0.80

Table 17. The optimized parameters of GKVM at temperature of 15°C of the BBTD

k_0 (MPa)	25000			
n°	**k_j (MPa)**	**□j (15°C) (MPa.s)**	**λ_j**	**τ_j**
1	1079000	4.53E-03	238290879.2	4.20E-09
2	805740	3.38E-02	23829087.92	4.20E-08
3	541400	2.27E-01	2382908.792	4.20E-07
4	500840	2.10E+00	2.38E+05	4.20E-06
5	213500	8.96E+00	23829.087	4.20E-05
6	105200	4.41E+01	2382.908	4.20E-04
7	80120	3.36E+02	238.29	4.20E-03
8	58576	2.46E+03	23.829	4.20E-02
9	20716	8.69E+03	2.3829	4.20E-01
10	16196	6.80E+04	0.2382	4.20E+00
11	10007	4.20E+05	0.02382	4.20E+01
12	8999	3.78E+06	0.002382	4.20E+02
13	5000	2,10E+07	0.0002382	4.20E+03
14	1000	4.20E+07	2.382E-05	4.20E+04
15	500	2.10E+08	2.382E-06	4.20E+05
16	215	9.02E+08	2.382E-07	4.20E+06
17	55	2.31E+09	2.382E-08	4.20E+07
18	25	1.05E+10	2.382E-09	4.20E+08
19	5	2.10E+10	2.382E-10	4.20E+09

Table 18. Statistical comparison between experimental TTSP master curve and model predictions

Model	Error (%)	RMSE (MPa)
Huet–Sayegh model	0.34	197.36
2S2P1D model	0.08	183.82
GKVM model	0.05	121.22

Table 19. Model ranking and selection for Engineering applications

Application	Most suitable model	Reason
Routine pavement design	Huet–Sayegh	Simple and accurate
Mechanistic pavement analysis	2S2P1D	Better representation of viscous effects
Advanced numerical simulation	GKVM	Highest accuracy
Finite element modelling	GKVM / 2S2P1D	Accurate time-dependent response
Material comparison studies	Huet–Sayegh	Limited parameters facilitate comparison

Table 20. Technical Standard of the New material tested « BBTD »

Material	E (MPa) at 10°C and 10 Hz	E (MPa) at 15°C and 10 Hz	E (MPa) at 28°C and 10 Hz	nu	ε_6 (µstrains) at 10°C and 25 Hz	b	S_N	k_c
BBTME	15150	13561	7641	0.35	106	- 0.2	0.223	1.1

Table 21. Technical Standard of the New material tested « BBTD » at multiple temperature and frequency

	Frequency (Hz)						
T (°C)	**5**	**10**	**15**	**20**	**25**	**30**	**35**
-10	22700	23564	24069	24427	24705	24932	25124
0	18339	19203	19708	20066	20344	20571	20764
10	14287	15150	15655	16014	16292	16519	16711
15	12295	13411	13757	14031	14114	14678	14895
20	10511	11374	11879	12237	12515	12743	12935
30	6983	7847	8352	8710	8988	9215	9407
40	3682	4545	5050	5408	5686	5914	6106

List of Figures

a) b)

Fig. 1. a) Rutting damage ; b) Fatigue cracking

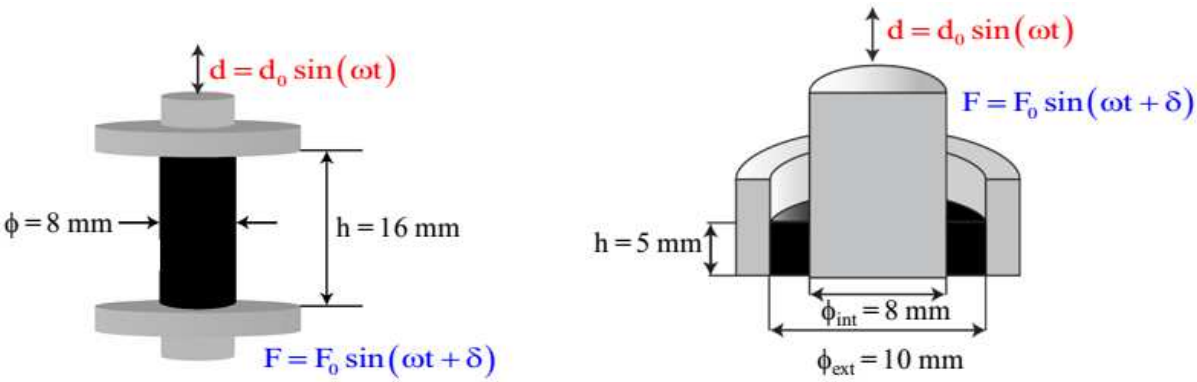


Fig. 2. Complex modulus testing of bitumen: left: tension–compression test; right: annular shear test [51]

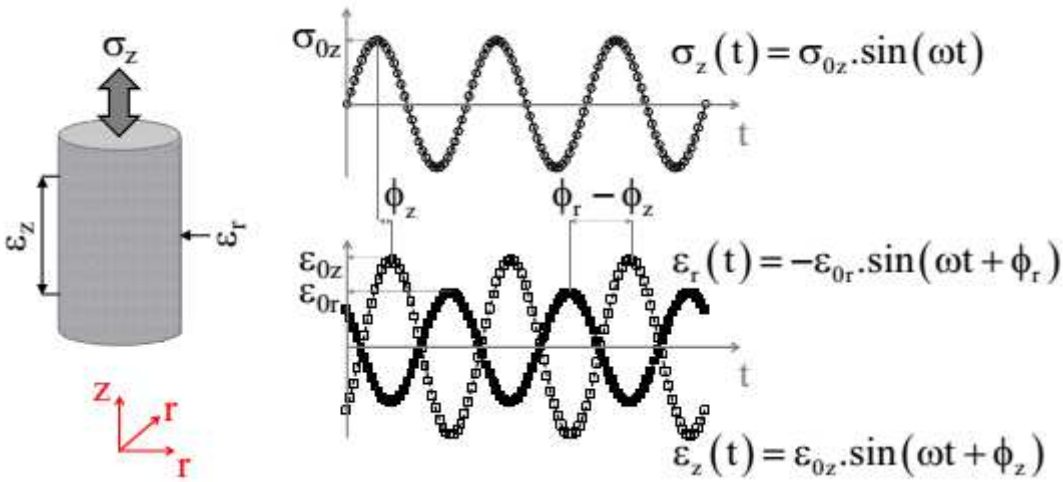


Fig. 3. Complex modulus measurement on a cylindrical specimen [51]

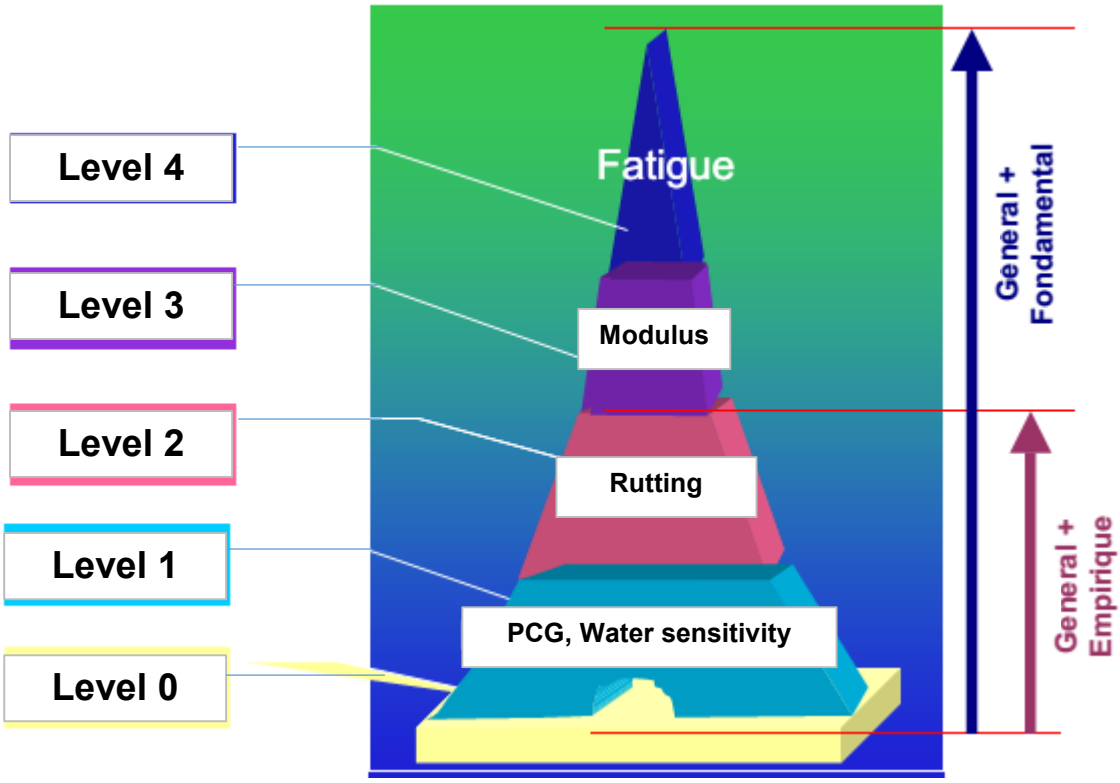


Fig. 4. Level of formulation using in this study according to NF P 98-150-1 [55]

Fig. 5. The mineral aggregates crushed gneiss from the Logbadjeck quarry (Cameroon)

Fig. 6. CECABASE® 300 mixture in the laboratory

Fig. 7. Polymer additive (PR PLAST S)

Fig. 8. Testing the bituminous concretes specimen on the giratory shear press

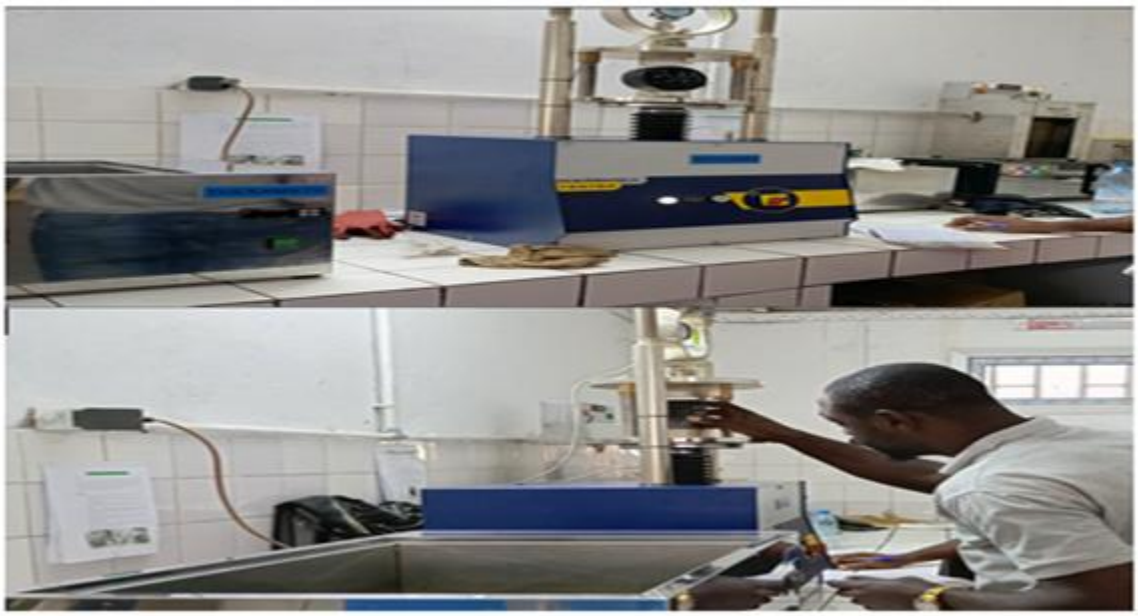


Fig. 9. Duriez Test of bituminous material with 0.3% CECABASE 300 Dosage

Fig. 10. Cylindrical specimens compacted in 100-mm diameter molds for ITSR test

Fig. 11. Rutting test on the slab asphalt concretes

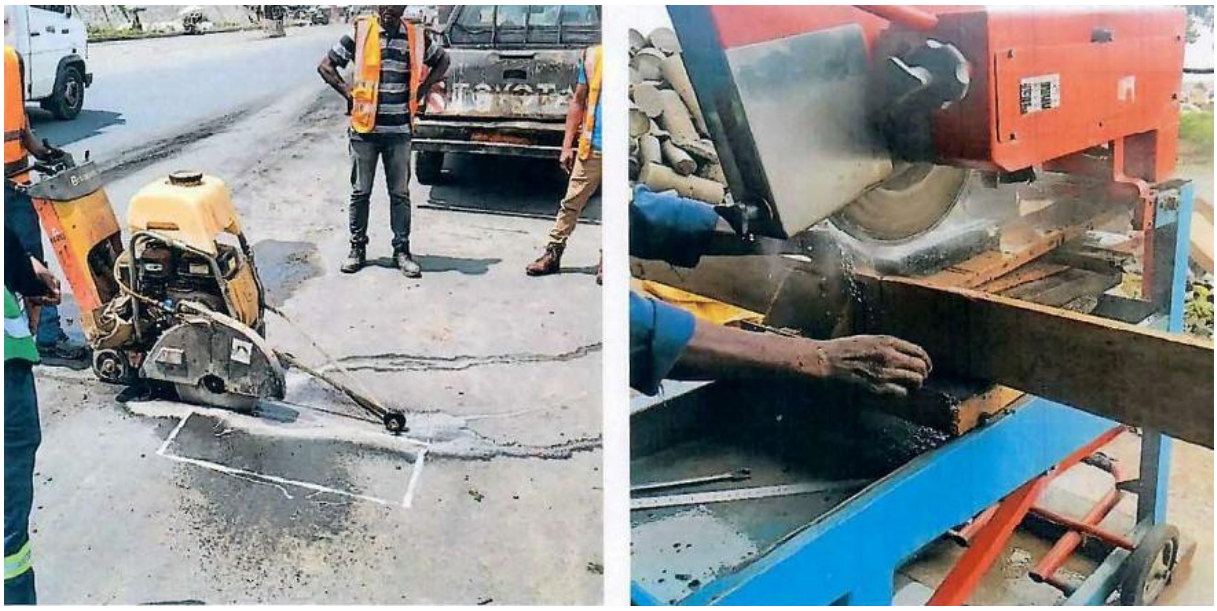


Fig 12. Trapezoidal specimens subsequently sawn from the slabs

Fig. 13. Stiffness test in the trapezoidal bituminous specimens in the laboratory

Fig. 14. Cylindrical cores extracted from the slabs and tested using the indirect tensile stiffness modulus method

Fig. 15. Trapezoidal specimens sawn from the slabs for fatigue testing

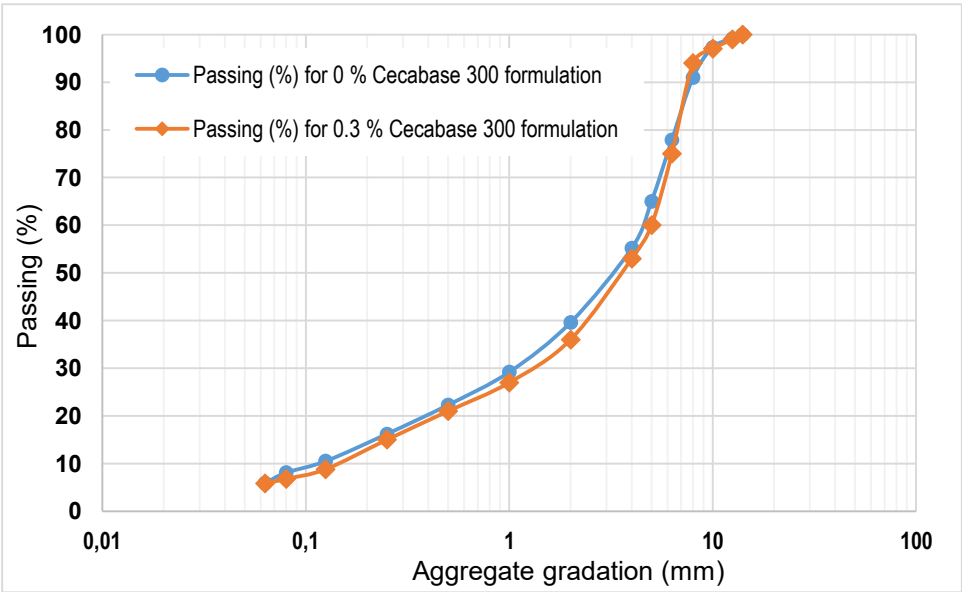
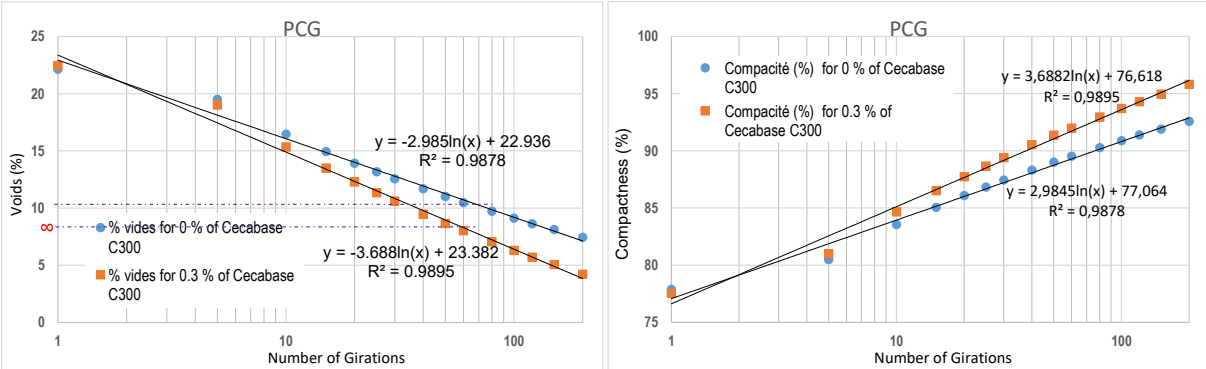


Fig. 16. Aggregate gradation curves of Logbadjeck quarry (Cameroon)



a) b)

Fig. 17. Gyratory compaction curves: a) Air voids versus number of gyrations; b) Compactness versus number of gyrations

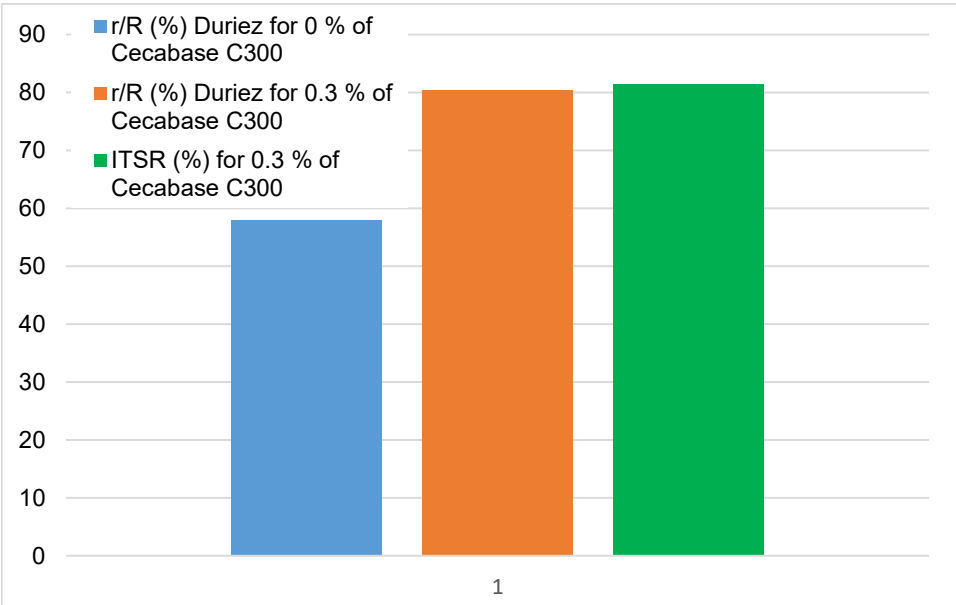


Fig. 18. Duriez and ITSR retained strength

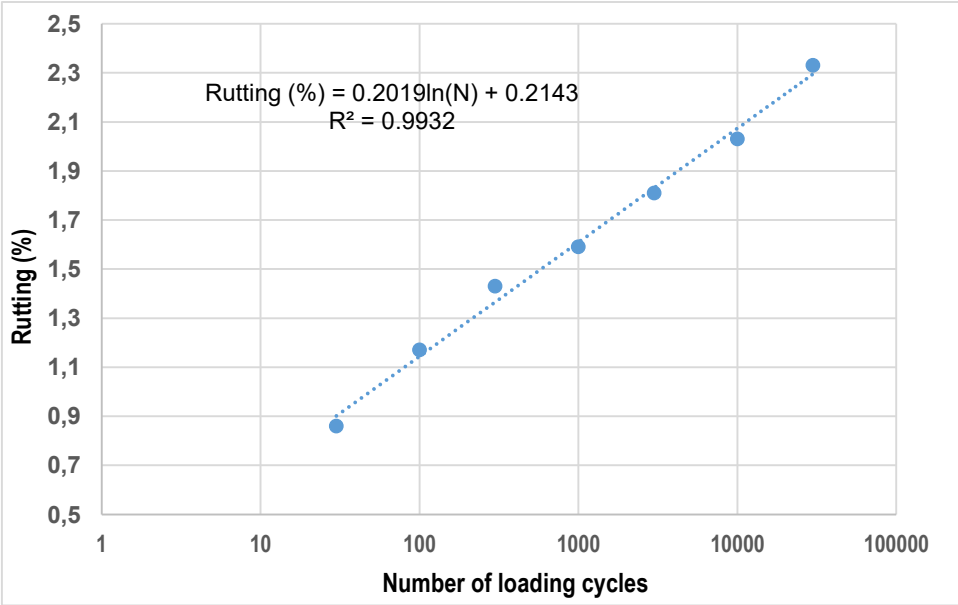


Fig. 19. Rut depth evolution

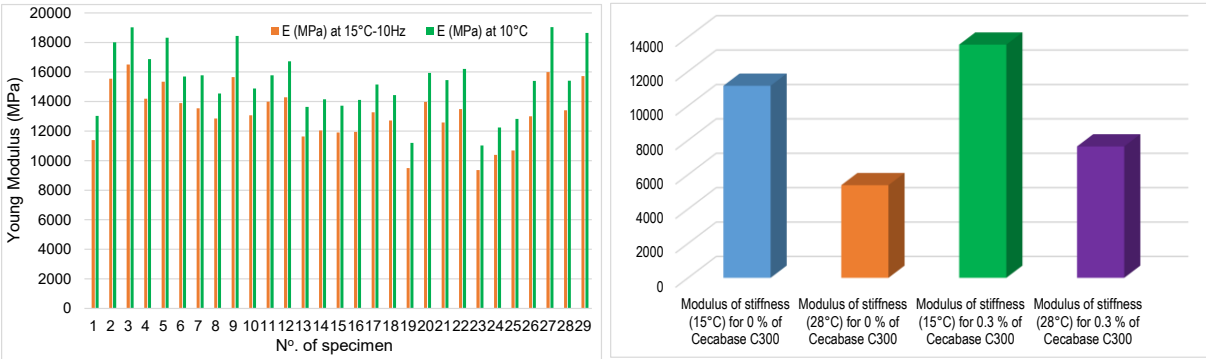


Fig. 20. Variability modulus distribution (15°C and 10°C)

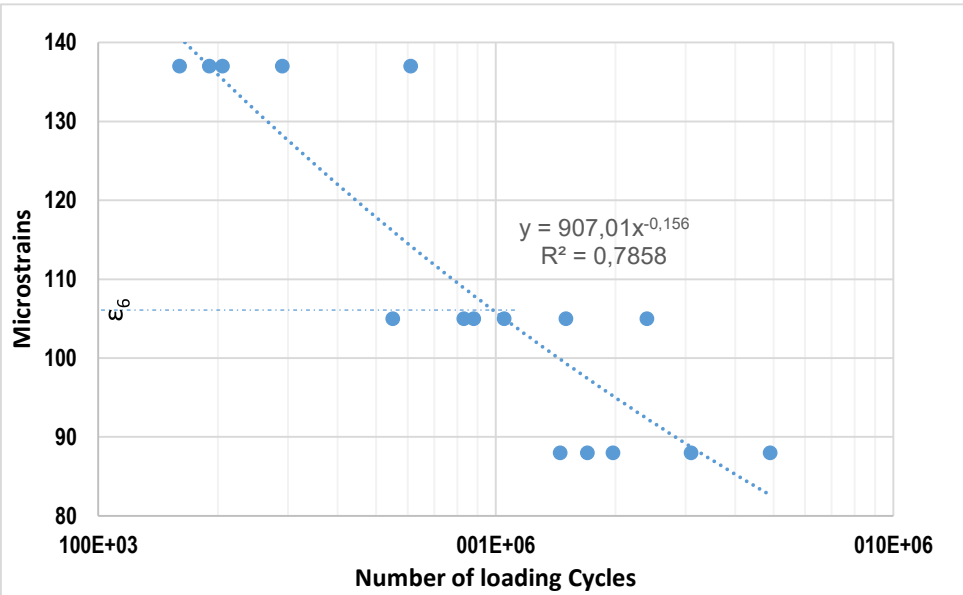


Fig. 21. Derivation the fatigue laws of the specimens

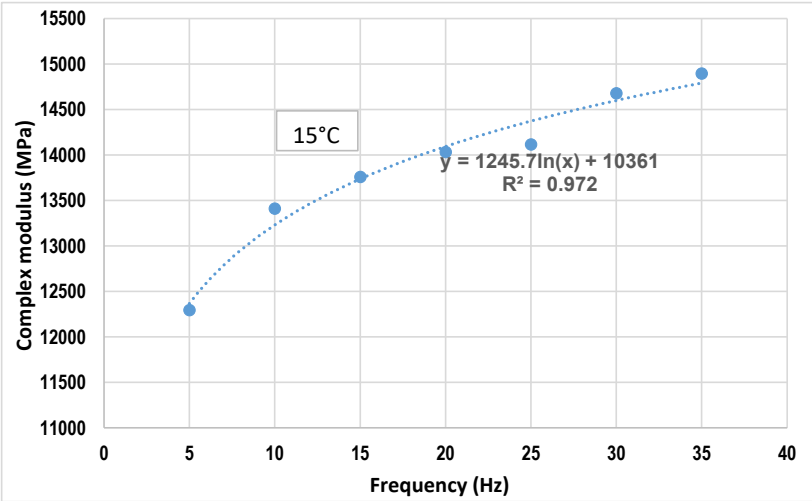
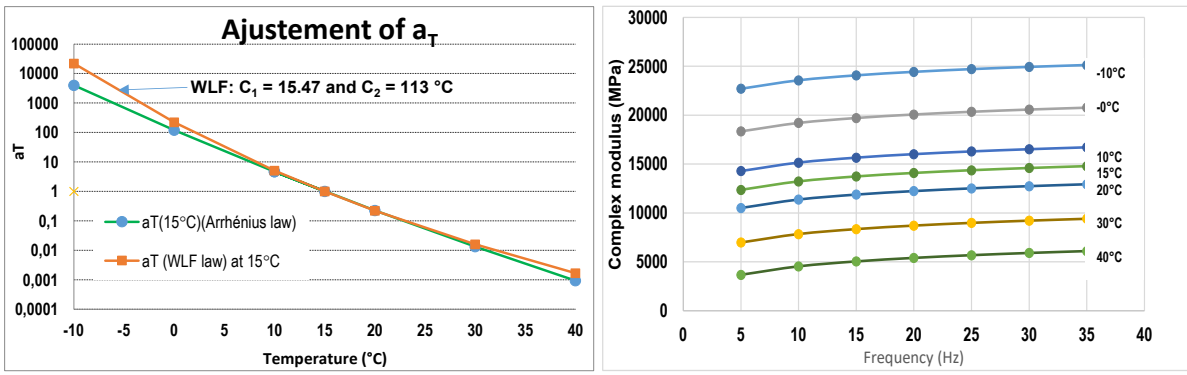


Fig. 22. Modulus evolution governed by the logarithm of loading frequency



a) **b)**

Fig. 23. a) Shift factors summarized ; b) Isothermal curve for the material

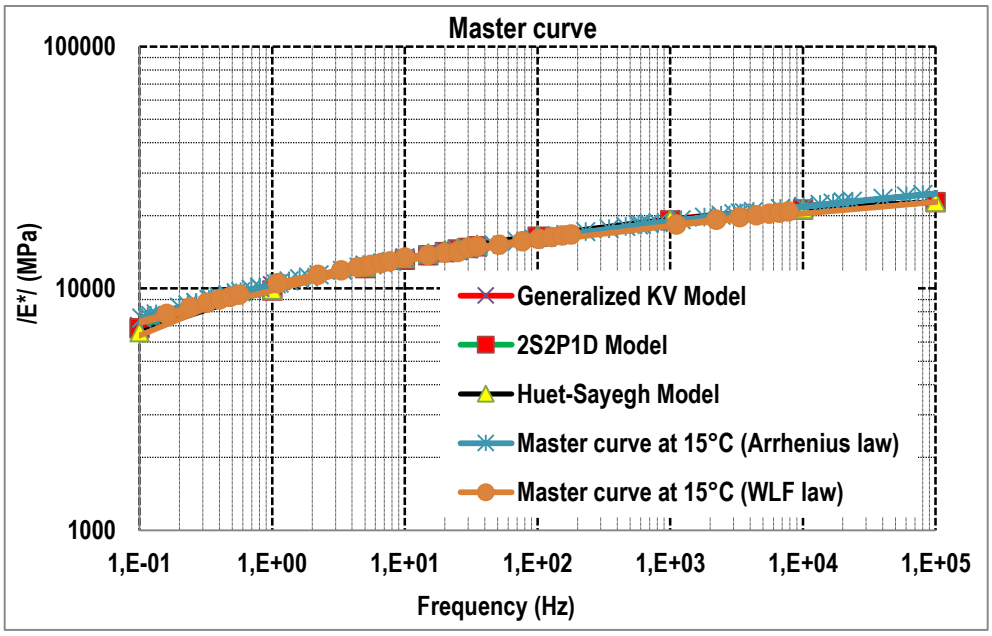


Fig. 24. Comparison master curve of constitutive models and experimental results