

Article

Performance-Based Optimization of Asphalt Mixtures for Tropical Pavement Structures: A Mechanistic and Fatigue Life Approach

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Abstract

Durable road infrastructure in tropical regions is challenged by severe climatic conditions and heavy traffic loads. This study investigates the influence of two high-performance binders and aggregate structure on the fatigue resistance of two asphalt concrete (AC) mixtures and examines their relationship with pavement mechanical response. The experimental program includes uniaxial compression and four-point bending fatigue tests to determine dynamic modulus and fatigue performance. A predictive model for constant-strain fatigue life was developed based on fatigue test results. Mechanistic analysis was conducted for different asphalt layer thicknesses to estimate internal strains used in fatigue life prediction. The results show that the high-toughness mixture (GT-13) exhibits a 12% to 14% higher dynamic modulus at high temperatures. The maximum tensile strain occurs at the bottom of the AC layer for a thickness of 7 cm. Compared to Beton bitumineux Semi-Grenu (BBSG 0/14), GT-13 improves fatigue life by over 5.4 times and reduces damage by about 87%, extending the equivalent theoretical design life beyond 28 years, achieving the lowest composite-index-based pavement evaluation.

Keywords: tropical pavements; PG binders; GT-13; BBSG 0/14; fatigue life; mechanical performance

1. Introduction

Sub-Saharan Africa has experienced rapid population growth and urbanization over recent decades, leading to increased demand for reliable transportation infrastructure. In countries such as the Republic of the Congo, road transport dominates, accounting for over 80% of freight and passenger movement [1]. Despite significant investments in road construction, pavement durability remains a major challenge. Premature failures, including cracking, potholes, and rutting, are frequently observed within a few years after construction, leading to increased maintenance costs and reduced serviceability. One of the primary causes of early pavement deterioration is the direct application of design methods and materials developed for temperate climates, particularly those from the French SETRA (Service d'études techniques des routes et autoroutes) and LCPC (Laboratoire Central des Ponts et Chaussées) [2], without adaptation [3,4]. However, tropical climates differ significantly due to severe environmental conditions, including higher average



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temperatures, accelerating oxidative aging of binders, elevated humidity, and alternations between intense rainfall and dry periods.

Beton bitumineux Semi-Grenu (BBSG 0/14) remains one of the most widely used surface courses, and has shown limited performance, under tropical conditions. Field observations indicate that pavements designed for 20 years often fail within 3 years [5]. This performance deficit is largely attributed to increasing traffic volumes combined with environmental effects exceeding the design assumptions of standard formulations [6,7]. This discrepancy raises a key scientific issue: the mismatch between material design assumptions and actual climatic and loading conditions.

The recent literature highlights that the lack of locally calibrated models, limited long-term performance data, and insufficient pavement temperature mapping represent major barriers to optimizing asphalt mixtures for sub-Saharan climates [4,8]. In addition, studies conducted in West Africa and analogous tropical regions have shown that high temperatures and humidity variations significantly affect mixture stability, resilience, and fatigue life [4,9]. To address these challenges, several strategies have been proposed, including enhanced pavement design methods, improved construction techniques, and binder modification [10–13]. Asphalt binder modification has been demonstrated to improve viscoelastic behavior [14–17], rutting resistance [18–21], moisture susceptibility [4,22], and fatigue resistance [23–26]. Previous studies [27,28] highlight that both binder properties and aggregate structure play a crucial role in asphalt performance. However, limited research has addressed this interaction under tropical conditions.

In sub-Saharan Africa, and particularly in the Republic of the Congo, pavement design and material selection are closely influenced by the public procurement system for road infrastructure projects. Public works contracts are mainly awarded through competitive bidding procedures, where initial construction cost remains a major evaluation criterion. This cost-driven decision process encourages the selection of lower-cost and locally available materials, which may not always provide optimal performance under severe tropical climatic conditions. The limited consideration of lifecycle cost analysis and long-term pavement performance further restricts the adoption of high-performance materials with potential durability benefits.

To address these limitations, this study adopts a performance-based, mechanistic pavement evaluation framework that incorporates key phases, including fatigue life estimation and composite index, integrating both economic and technical factors into a single metric, to assess the long-term behavior of two mixtures. A conventional BBSG 0/14 and a high-toughness (GT-13) asphalt mixture were evaluated using an SBS-modified PG76-22 and a high-viscosity and high-elasticity PG94 modified binder.

The experimental program included determination of the optimum binder content (OBC), dynamic modulus testing, and four-point bending fatigue testing. In addition to experimental characterization, fatigue performance was further analyzed through predictive modeling. Based on the results obtained from four-point bending fatigue results, a fatigue life prediction model was developed by modifying the constant-strain fatigue model originally proposed by the Road Engineering Research Institute of South China University of Technology using conventional bitumen. To link laboratory results to field performance, the mechanical properties were incorporated into pavement simulations using the multilayer elastic program Alizé-LCPC (Laboratoire Central des Ponts et Chaussées). A case-study analysis was conducted on the RN2 national road in the Republic of the Congo, a critical piece of infrastructure supporting domestic freight logistics and regional economic integration. The pavement structure analysis incorporated variations in key input parameters, including real traffic, climatic conditions, pavement composition, and material properties.

Finally, the fatigue life performance was predicted based on the critical horizontal strain at the bottom of the bituminous layer.

Therefore, the main objective of this study is to optimize asphalt surface courses for tropical climates based on performance criteria. Specifically, it aims to (1) compare BBSG 0/14 and GT-13 mixtures; (2) evaluate the effects of PG76-22, PG94, and aggregate structure in the performance of asphalt mixtures; (3) establish relationships between mechanical properties and pavement response with respect to fatigue life performance; and (4) consider economic aspects, including the use of composite indices (CLR—Cost-Life Ratio; MEI—Material Efficiency Index) for pavement performance evaluation. Figure 1 illustrates the methodology algorithm. The results are intended to support the adaptation and optimization of asphalt mixtures for sustainable road infrastructure in the Republic of the Congo and similar regions.

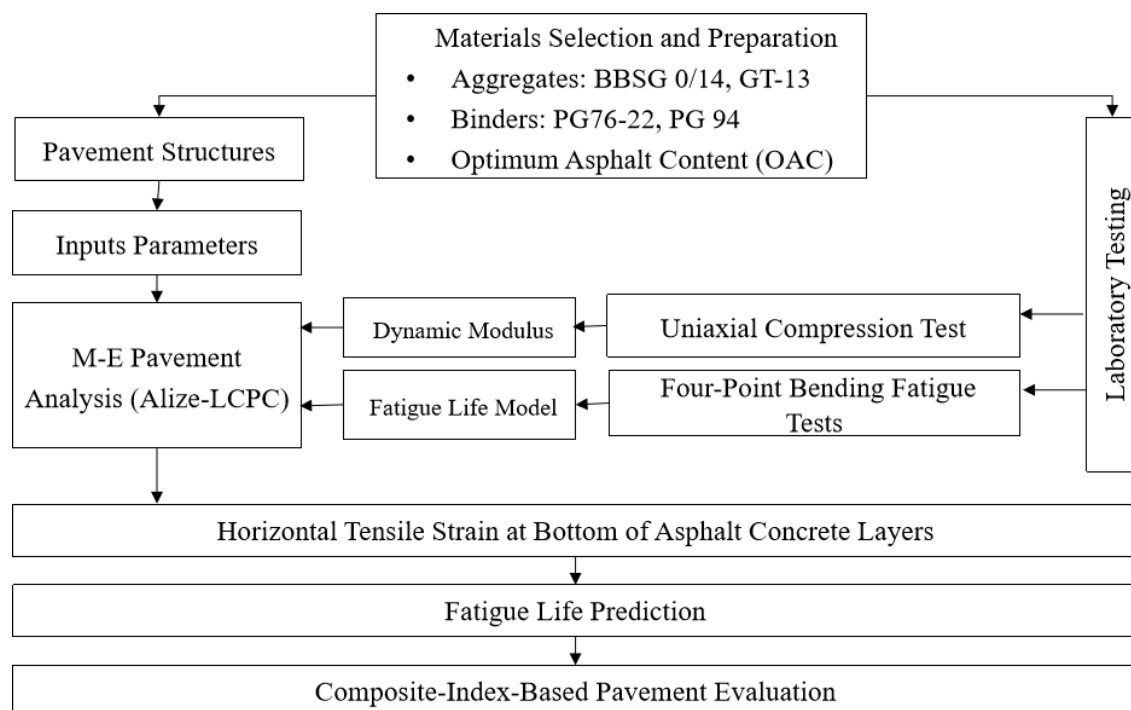


Figure 1. The research methodology algorithm.

2. Materials and Methods

2.1. Raw Materials and Gradation Design

2.1.1. Aggregates and Mineral Fillers

The aggregate gradation comprised limestone fractions of 0–3 mm, 5–10 mm, and 10–15 mm sourced from Qisheng Quarry (Wuzhou, Guangxi, China). The filler was limestone powder produced by Quanfa Stone Factory (Foshan, Guangdong, China). The limestone materials demonstrated stable physical properties and superior asphalt adhesion characteristics, contributing to enhanced performance of the asphalt mixtures. Notably, similar limestone resources are abundantly available in the Republic of the Congo, suggesting excellent suitability for local pavement applications with significant economic advantages. The technical properties of the aggregates and powder are shown in Tables 1–3. The specifications were met for all data [29].

Table 1. Technical properties of coarse aggregate.

Test Items	Requirements	Results	Test Method
Crushing value (%)	≤ 26	23.1	JTG 3432 T0316
LA abrasion loss value (%)	≤ 28	22.3	JTG 3432 T0317
Apparent relative density	≥ 2.6	2.728	JTG 3432 T0304
Water absorption (%)	≤ 2	0.391	JTG 3432 T0304
Flat/elongated particles content (%)	≤ 12	6.1	JTG 3432 T0312

Table 2. Technical properties of fine aggregate.

Test Items	Requirements	Results	Test Method
Apparent relative density	≥ 2.5	2.700	JTG 3432 T0328
Soundness (>0.3 mm) (%)	≤ 12	8	JTG 3432 T0340
Sand equivalent (%)	≥ 60	65	JTG 3432 T0334

Table 3. Technical properties of powder.

Test Items	Requirements	Results	Test Method
Apparent specific gravity	≥ 2.5	2.672	JTG 3432 T0352
Moisture content (%)	≤ 1	0.2	JTG 3432 T0359
Hydrophilic coefficient	< 1	0.7	JTG 3432 T0353
Plasticity index (%)	< 4	2.8	JTG 3432 T0354

2.1.2. Modified Asphalt Binders

The Shell SBS (Styrene-Butadiene-Styrene)-modified asphalt PG76-22 binder was supplied by Guangdong Xinyue Asphalt Co., Ltd. (Guangzhou, China), while the PG94 high-viscosity and high-elasticity modified asphalt binder was provided by Central Fortune Creation Technology Group Co., Ltd. (Foshan, China). The technical properties of both asphalt binders are presented in Table 4. The specifications for pavement asphalt binders were met for all data [30,31].

Table 4. Technical properties of modified asphalt binders.

Test Items	PG76-22		PG94		Test Method
	Requirements	Results	Requirements	Results	
25 °C Penetration (0.1 mm)	40~80	54	30~60	41	T0604
Softening point (°C)	≥ 76	79	≥ 92	93.5	T0606
Solubility in trichloroethylene (%)	≥ 99	99.8	≥ 99	99.5	T0607
Flash point (°C)	≥ 230	337	≥ 260	318	T0611
60 °C Dynamic viscosity (Pa·s)	>30,000	65,462	>580,000	>580,000	T0620
25 °C Elastic recovery (%)	≥ 90	96	≥ 96	99	T0662
Properties of the RTFOT residue (%)					
Mass change	± 1	−0.023	± 1	−0.048	T0609
Residual penetration ratio (25 °C)	≥ 65	78.4	≥ 70	82.9	T0604

2.1.3. GT-13 Gradation Design

According to the GT-13 gradation limits specified for high-toughness asphalt wearing courses [31,32], with 4.75 mm selected as the key sieve for coarse aggregate skeleton structure, three initial gradation variants with varying coarseness were designed by adjusting the mineral aggregate proportions. The passing rates of the key sieve for all three gradations were controlled within $\pm 3\%$ of the median value in the specification range. Through computational analysis and gradation curve optimization, the blended gradations and individual fraction contents of coarse/fine aggregates were determined as shown in Figure 2.

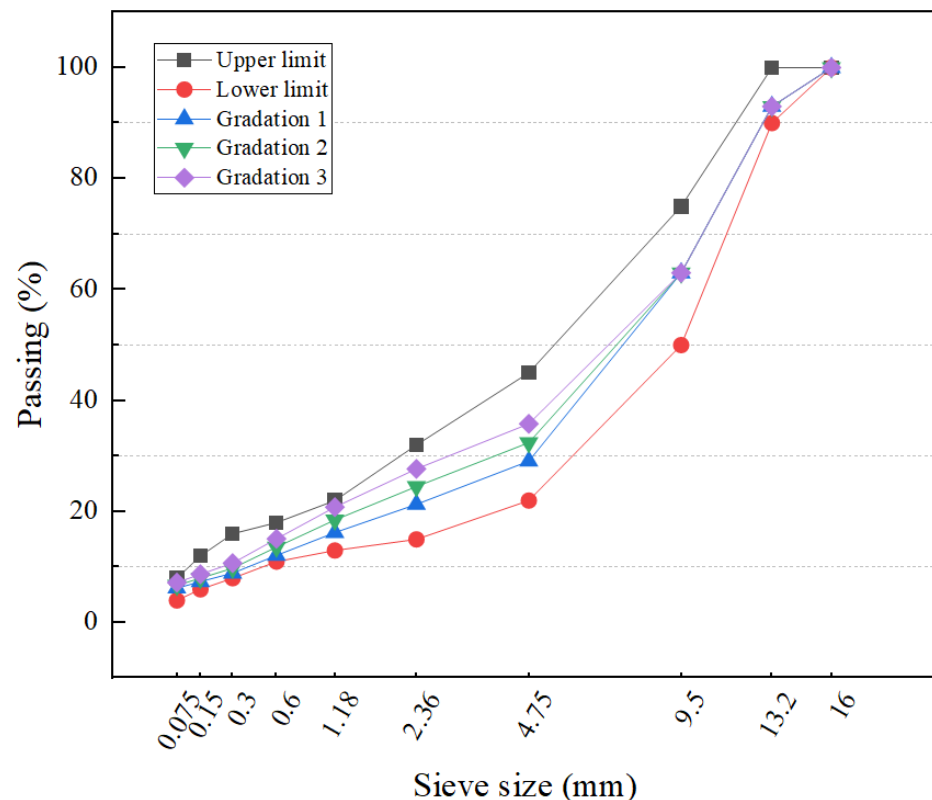


Figure 2. GT-13 gradation curves and grading envelope.

The percentage voids in the coarse aggregate skeleton (VCA_{DRC}) of each gradation were measured according to the rodded bulk density test [29]. A cylindrical measure was horizontally positioned on a laboratory bench. Samples were introduced in three equal layers, each rodded 25 times from edges to center. The percentage voids in the coarse aggregate of asphalt mixture (VCA_{mix}) were measured as described in [30]. Four Marshall specimens were prepared for each gradation, as described in [33]. The PG94 asphalt binder was preheated to 185–195 °C for 3–5 h, while aggregates were heated to 195 °C for 4 h or more prior to mixing. The mixture was prepared at 195 ± 5 °C and compacted with 75 blows on each side, with an initial asphalt binder content of 6.0%. Specimens' bulk specific gravity (BSG) and theoretical maximum specific gravity [33] were used to derive volumetric parameters, such as air voids (AV), voids in mineral aggregate (VMA), and voids filled with asphalt (VFA). Marshall Stability (MS) and flow value (FV) were determined as described in [33]. The test results are detailed in Table 5.

Table 5. GT-13: each gradation group's design Marshall test results.

Gradations Design	BSG	AV (%)	VMA (%)	VFA (%)	MS (kN)	FV (0.1 mm)	VCA _{mix} (%)	VCA _{DRC} (%)
Gradation1	2.392	3.5	15.4	77.4	14.77	3.1	40.1	43.03
Gradation 2	2.425	2.1	14.2	85.1	13.95	4.9	42.2	42.68
Gradation 3	2.431	1.9	14.0	86.8	15.25	6.7	44.8	43.71

Notes: BSG (bulk specific gravity), AV (air voids), VMA (voids in mineral aggregate), VFA (voids filled with asphalt), MS (Marshall Stability) and FV (flow value), VCA_{DRC} (voids in the coarse aggregate skeleton), VCA_{mix} (voids in the coarse aggregate of asphalt mixture).

As shown in Table 5, Gradations 1 and 2 satisfied the $VCA_{mix} < VCA_{DRC}$ criterion [30], while Gradation 3 failed ($VCA_{mix} > VCA_{DRC}$). Gradation 1 and Gradation 2 exhibited acceptable aggregate skeleton characteristics, with VCA_{mix} values of 40.1% and 42.2%, respectively. However, Gradation 1 presented a lower VCA_{mix} and a higher VCA_{DRC} (43.03%) compared with Gradation 2 (42.68%), indicating a more favorable aggregate skeleton. In addition, Gradation 1 achieved an appropriate air void content ($AV = 3.5\%$) and a higher VMA (15.4%), compared with Gradation 2 ($AV = 2.1\%$ and $VMA = 14.2\%$). The higher VMA of Gradation 1 provides more available space for binder accommodation, which is beneficial for maintaining adequate binder coverage and mixture durability under hot tropical conditions. Meanwhile, the slightly higher air void content ensures sufficient internal structure without excessive densification. Therefore, considering the combined effects of aggregate skeleton characteristics and volumetric properties, Gradation 1 was selected as the final GT-13 gradation design. Aggregate fractions and gradation limits are provided in Table 6.

Table 6. GT-13 gradation limits.

	Cumulative Passing (%)									
Sieve size (mm)	16	13.2	9.5	4.75	2.36	1.18	0.6	0.3	0.15	0.075
Upper limit	100	100	75	45	32	22	18	16	12	8
Lower limit	100	90	50	22	15	13	11	8	6	4
Blended Gradation 1	100	93	63	29.1	21.3	16.2	12.1	8.9	7.4	6.2
Blended Gradation 2	100	93	63	32.4	24.5	18.5	13.6	9.8	8.0	6.7
Blended Gradation 3	100	93	63	35.7	27.7	20.8	15.1	10.7	8.7	7.2

2.1.4. BBSG 0/14 Gradation Design

The BBSG 0/14 mixture was designed in compliance with [34,35], with target stiffness values of $E \geq 7000$ MPa (15 °C, 10 Hz) and $E \geq 2400$ MPa (28 °C, 10 Hz) [36]. The blended aggregate gradation conformed to the sieve limits specified in [34]. The grading curve and envelope are presented in Figure 3, while the proportions of aggregate fractions and the corresponding gradation limits are detailed in Table 7.

Table 7. BBSG 0/14 gradation limits.

	Cumulative Passing (%)									
Sieve size (mm)	16	14	10	6.3	4	2	0.5	0.25	0.125	0.063
Upper limit	100	100	100	70	50	35	20	15	10	8
Lower limit	100	90	80	50	35	20	10	5	4	4
Blended gradation	100	98.1	88.3	57.7	39.4	27.8	15	10.6	8.6	6.3

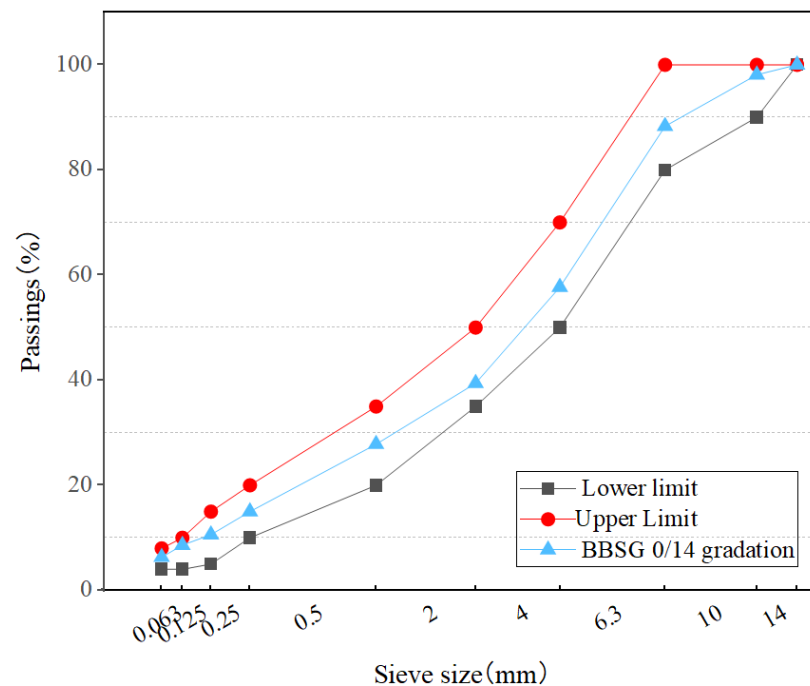


Figure 3. BBSG 0/14 gradation curves and grading envelope.

2.1.5. Optimum Binder Content

The optimum binder content (OBC) for GT-13 was determined using Marshall testing, while the BBSG 0/14 mix employed the richness coefficient method.

1. Marshall Test Method

Following the GT-13 gradation design, four replicate Marshall specimens were fabricated at each of five asphalt content (PG94) levels, which were varied at increments of 0.2%. The PG94 was preheated to 185–195 °C for 3–5 h, while aggregates were heated to 195 °C for 4 h or more prior to mixing. The mixture was prepared at 195 ± 5 °C, and a total of 20 Marshall specimens were compacted with 75 blows on each side. The BSG of the specimens was determined by using the saturated-surface-dry (SSD) method [33], while the theoretical maximum specific gravity was empirically calculated. The experimental results are summarized in Table 8.

Table 8. Experimental results of Marshall test method (average of 4 specimens by mixture).

Asphalt Content (%)	BSG	AV (%)	VMA (%)	VFA (%)	MS (KN)	FV (0.1 mm)
5.6	2.379	4.5	15.6	71.0	12.33	2.6
5.8	2.386	4.0	15.5	74.5	14.19	2.8
6.0	2.392	3.5	15.4	77.4	14.77	3.1
6.2	2.395	3.1	15.5	80.0	13.97	3.6
6.4	2.394	2.9	15.6	81.5	11.33	4.2

Notes: BSG (bulk specific gravity), AV (air voids), VMA (voids in mineral aggregate), VFA (voids filled with asphalt), MS (Marshall Stability), and FV (flow value).

The Marshall test parameters were analyzed as functions of asphalt content, with their respective values plotted on the ordinate against the asphalt content on the abscissa, as presented in Figure 4.

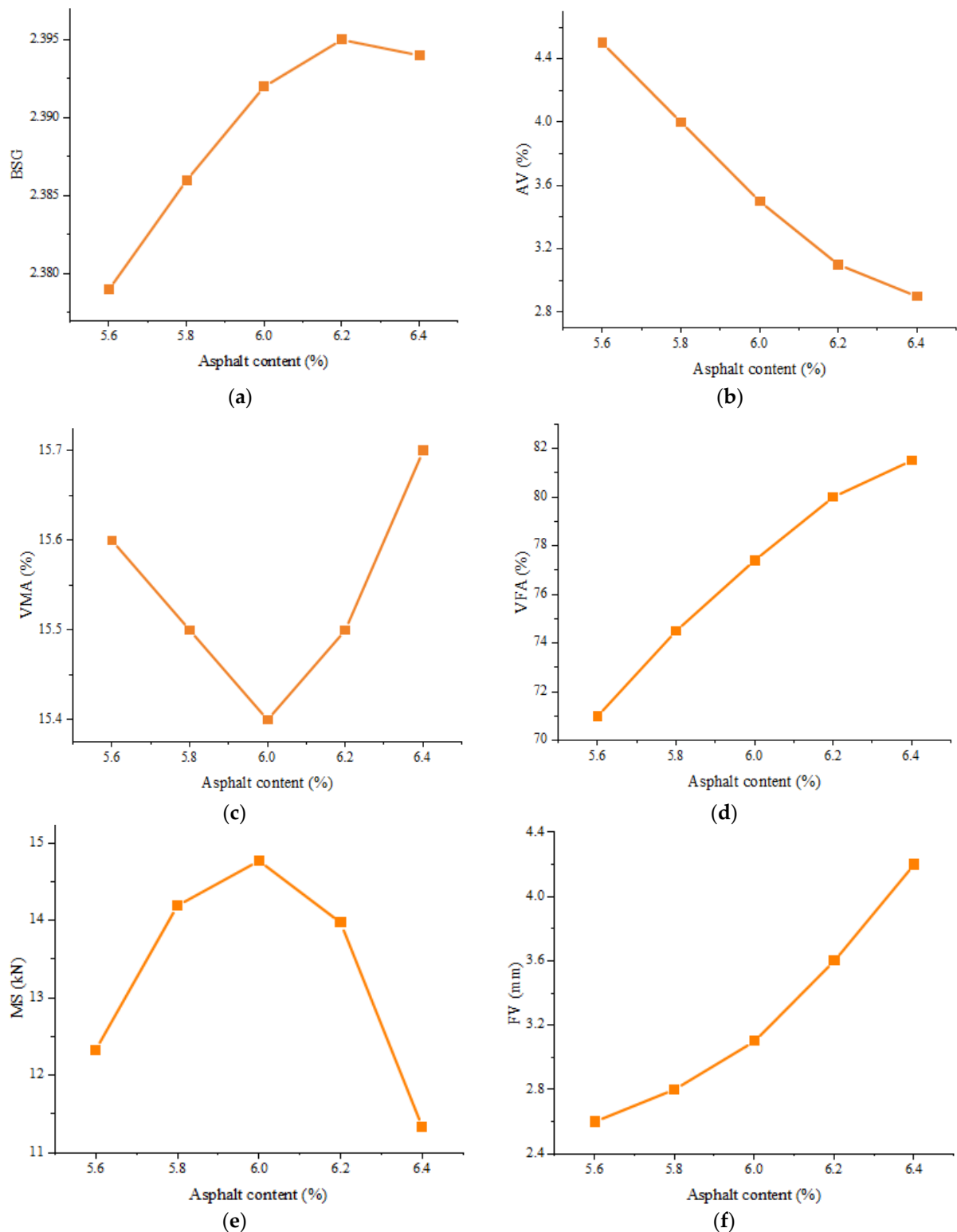


Figure 4. Relationships between asphalt content and volumetric/Marshall parameters. (a) BSG (bulk specific gravity); (b) AV (air voids); (c) VMA (voids in mineral aggregate); (d) VFA (voids filled with asphalt); (e) MS (Marshall Stability); (f) FV (flow value).

Based on the curve trends and comprehensive analysis of characteristic parameter values against technical specifications [30], the OBC for the PG94 + GT-13 mixture was determined to be 5.9%.

2. Richness coefficient method

The blended gradation curve of BBSG 0/14 was used to determine the key sieve-passing rates required by the richness coefficient method (Equation (1)), with the minimum richness coefficient value then applied to calculate the initial asphalt content.

$$K = \frac{TL_{\text{ext}}}{\alpha \sqrt[5]{\Sigma}} \quad (1)$$

$$\Sigma = \frac{0.25G + 2.3S + 12s + 150f}{100} \quad (2)$$

where K is the richness coefficient; TL_{ext} is the asphalt content; $\alpha = \frac{2.65}{\rho_G}$ is the aggregate density correlation factor; Σ is the specific surface area, expressed in square meters per kilogram, determined by Equation (2); $G = 42.3\%$ is the coarse aggregate proportion (>6.3 mm); $S = 47.1\%$ is intermediate aggregate proportion (6.3 – 0.25 mm); $s = 4.3\%$ is the fine aggregate proportion (0.25 – 0.063 mm); $f = 6.3\%$ is the filler proportion (<0.063 mm); and ρ_G is the specific gravity of the aggregate.

The initial target asphalt content of 5.29% was calculated using the richness coefficient ($K = 3.3$), as described in [34]. This standard requires validating this content with a Superpave Gyratory Compactor (SGC) at 80 gyrations to achieve a mix with 4–9% air voids. The OBC is defined as the value that best satisfies these air void requirements. The BBSG 0/14 was produced with three different bitumen contents: 5.3%, 5.6%, and 5.9%. The PG94 asphalt binder was preheated to 185–195 °C for 3–5 h, while aggregates were heated to 195 °C for 4 h or more prior to mixing. The specimens of 150 mm $\times$ 60 mm were prepared at 195 $\pm$ 5 °C and compacted with 80 gyrations. The mixture's maximum density and air void content were determined as described in [37,38], respectively. A linear regression analysis of the relationship between asphalt content and air voids was performed to determine the OBC. The obtained OBC was 5.57%, corresponding to the target air void content of 5% for the BBSG 0/14 mixture.

2.2. Methods

2.2.1. Uniaxial Compression Testing

The dynamic modulus (E) was determined in accordance with [39], using uniaxial compression testing on cylindrical specimens with OBC. Initial 150 $\times$ 170 mm specimens were compacted via SGC and cored to 100 $\times$ 150 mm dimensions for subsequent testing. Twelve (12) specimens were tested under the same previous conditions: three specimens per mixture and temperature were tested using the Universal Testing Machine (UTM-130). The average of the three results was used as the final value. The tests were conducted at three temperatures (15 °C, 20 °C, and 28 °C) under 10 Hz and 25 Hz loading frequencies. The dynamic modulus of all specimens was determined according to Equation (3). Figure 5 shows the compacted specimens during the uniaxial compression test.

$$|E^*(\omega)| = \frac{|\sigma^*|}{|\epsilon^*|} \quad (3)$$

where

$|E^*(\omega)|$ is the dynamic modulus for frequency ω (MPa);

$|\sigma^*|$ is the stress magnitude (kPa);

$|\epsilon^*|$ is the average strain magnitude.

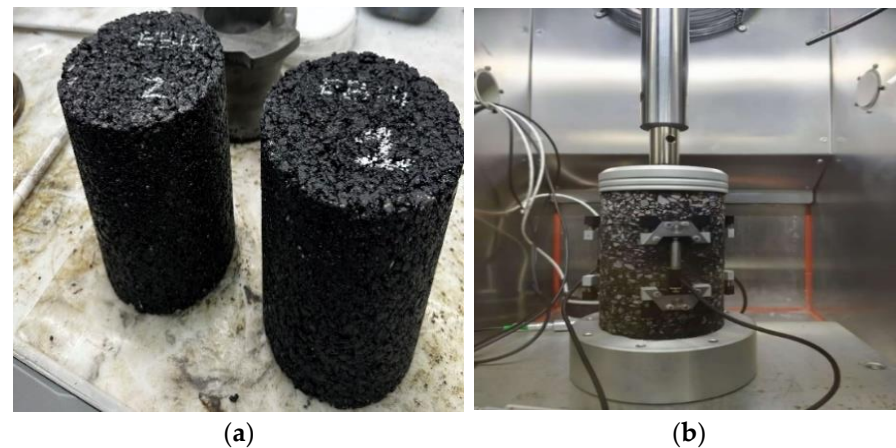


Figure 5. Uniaxial compression testing: (a) specimens after compaction; (b) specimen during testing.

2.2.2. Four-Point Bending Fatigue Tests

This study employed four-point bending fatigue tests [40] conducted on precisely dimensioned asphalt beams ($380 \times 50 \times 63$ mm) to determine fatigue life (N_f50) under simulated service conditions. Seventy-two (72) valid beams were tested under the fatigue loading conditions. Three replicate beams were prepared for each combination of mixture type, binder type, strain level, and temperature condition, and the average value of the three replicates was used as the representative fatigue life. The fatigue tests were conducted at five strain levels (300, 400, 500, 800, and 1000 $\mu\epsilon$), with temperature conditions selected according to the loading severity: 300 $\mu\epsilon$ and 400 $\mu\epsilon$ were tested at 10 °C and 15 °C, respectively; 500 $\mu\epsilon$ was evaluated at both 20 °C and 28 °C; 800 $\mu\epsilon$ and 1000 $\mu\epsilon$ were tested at 28 °C. All fatigue tests were performed at a loading frequency of 25 Hz. This frequency, although higher than typical traffic-induced loading frequencies, was selected according to the standardized testing procedures specified in [35,36]. The use of this standardized frequency ensures consistency with pavement material characterization procedures, improves test reproducibility, and enables direct comparison of fatigue performance between the two mixtures under identical laboratory conditions. Figure 6 shows the beam specimens and the four-point bending fatigue test.

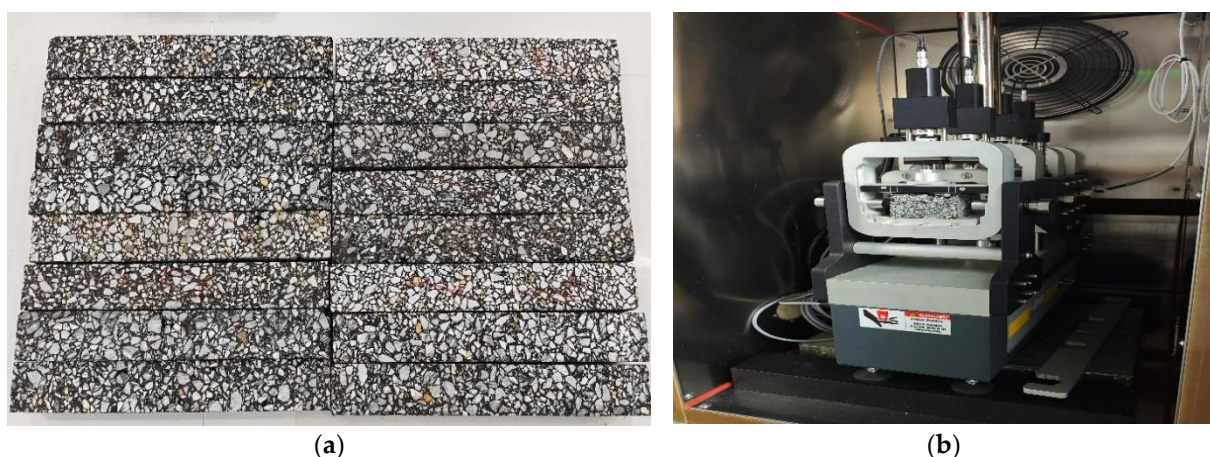


Figure 6. Four-point bending fatigue tests: (a) beam specimens; (b) specimen during testing.

2.2.3. Fatigue Life Prediction Approach

The fatigue life (N_f) was predicted using constant-strain fatigue models established from the experimental results of high-performance asphalt mixtures. The following calculation was used to evaluate the fatigue damage (D) sustained by the surface layer. In this approach, the cumulative number of heavy goods vehicles (N_{PL}) over the design life was converted into an equivalent number of reference axle loads (NE) using the average traffic aggressiveness coefficient (CAM), according to Equation (4) [35]. The fatigue damage was then calculated as the ratio between NE and N_f , as expressed in Equation (5).

$$NE = N_{PL} \times CAM \quad (4)$$

For this case, the CAM for bituminous materials is equal to 0.8, while for subgrade it is equal to 1 [35].

$$D = \frac{NE}{N_f} \quad (5)$$

2.2.4. Composite-Index-Based Pavement Evaluation

Beyond technical considerations, economic factors are equally critical to the comprehensive assessment. The evaluation employed two composite indices to integrate economic and technical factors into a single metric: The first is the Cost-Life Ratio (CLR), relating construction cost to service life (Equation (6)). The second is the Material Efficiency Index (MEI), which incorporates service life, construction cost, and the volume of imported material via layer thickness (H_m) (Equation (7)) [41].

$$CLR = \frac{\text{Construction Cost} [XAF/m^2]}{\text{Service Life} [N_f]} \quad (6)$$

Here, XAF (ISO 4217) denotes the Franc of Financial Cooperation in Central Africa.

A lower CLR points to a more cost-effective pavement option. This indicator reflects material usage indirectly, since the cost of materials is a component of the total construction cost.

$$MEI = \frac{\text{Construction Cost} [XAF/m^2] \times \text{Layer Thickness} [m]}{\text{Service Life} [N_f]} \quad (7)$$

The MEI unit is $XAF/m^2/N_f$, representing the combined cost and material requirements of each N_f . A lower MEI value signifies a more favorable balance among cost, material use, and lifetime for a given pavement configuration.

To fulfill the objectives of this study, the cost information from the Congolese Road Maintenance Authority (DGER) was utilized.

3. Results and Discussion

3.1. Dynamic Modulus

Dynamic modulus (E) test results are presented in Table 9. The comparison of E values at three testing temperatures for all of the mixes is also presented in Figure 7. The dynamic modulus of the four asphalt concrete (AC) mixtures significantly reduced with increasing temperature. This indicates the adverse effects of temperature on AC mixtures. As the temperature increases, the asphalt binder softens, resulting in a decrease in the E values of the mixtures. Compared to mixtures made with PG76-22, the mixture with PG94 had the lowest stiffness value across all testing temperatures. At each test temperature and loading frequency, GT-13 had the highest elastic modulus value among the two asphalt binders

compared to BBSG 0/14. When formulated with PG76-22, GT-13 showed dynamic modulus increases of 2.4%, 3.3%, and 14.6% at 15 °C, 20 °C, and 28 °C, respectively, at 10 Hz; at 25 Hz, the increases were 2.43%, 3.8%, and 14.23% for the same temperatures. With PG94, the corresponding increases at 10 Hz were 6.91%, 6.93%, and 13.21%, while at 25 Hz they were 6.92%, 8.03%, and 12.3%, respectively. Specifically, at high temperatures, the dynamic modulus of the GT-13 mixture exceeds that of the BBSG 0/14 mixture by approximately 14%, indicating the potential of GT-13 to improve rutting resistance in asphalt pavements. The thermal susceptibility analysis indicates that GT-13 exhibits better modulus retention, with a reduction of 50.5–52.7% between 15 °C and 28 °C, compared with 53.3–57.8% for BBSG 0/14. The improved thermomechanical stability of GT-13 can be attributed to the combined effects of aggregate gradation and binder content, which contribute to its suitability for high-temperature service conditions. In addition, the variability in the dynamic modulus results was evaluated using the coefficient of variation (CV). The CV values ranged from 0.55% to 2.68% for BBSG 0/14 mixtures and from 0.69% to 1.93% for GT-13 mixtures, indicating limited dispersion among the measured values.

Table 9. Dynamic modulus test results.

AC Mixtures		Frequencies Hz	E (15 °C) MPa	E (20 °C) MPa	E (28 °C) MPa
PG94	BBSG 0/14	10	7192 ± 72	5774 ± 55	3361 ± 90
	GT-13		7689 ± 83	6174 ± 91	3805 ± 68
	BBSG 0/14	25	8685 ± 95	7015 ± 89	4251 ± 80
	GT-13		9286 ± 87	7578 ± 78	4772 ± 92
PG76-22	BBSG 0/14	10	10,102 ± 66	7051 ± 48	4266 ± 81
	GT-13		10,340 ± 71	7285 ± 59	4887 ± 76
	BBSG 0/14	25	11,964 ± 66	9105 ± 74	5522 ± 95
	GT-13		12,255 ± 85	9451 ± 81	6308 ± 54

3.2. Fatigue Resistance

Table 10 presents the fatigue life test results for BBSG 0/14 and GT-13 mixtures, each formulated with both PG76-22 and PG94, under identical testing conditions employing a constant strain level, controlled temperature, and a loading frequency of 25 Hz. Fatigue resistance can be observed to decrease as both temperature and deformation are increased. The results demonstrate that the PG94 improves fatigue resistance by 28–39% compared to PG76-22 for both asphalt types. Under moderate conditions (300–400 µε strain, 10–15 °C), both mixtures exhibit comparable fatigue resistance, sustaining approximately 2–3 million load cycles. However, GT-13 demonstrates higher performance under severe loading conditions (500–1000 µε strain and 28 °C), displaying 11.63 times greater fatigue resistance at 500 µε, 5.45 times at 800 µε, and 4.19-fold enhanced durability at 1000 µε. Notably, GT-13 maintains higher thermal stability, showing only a 6–17% reduction in fatigue resistance between 20 and 28 °C, compared to the 56–59% degradation observed in BBSG 0/14. These results indicate fundamentally different damage tolerance mechanisms, with GT-13 exhibiting high resistance to micro-crack propagation and enhanced energy dissipation capabilities under combined thermal–mechanical stresses.

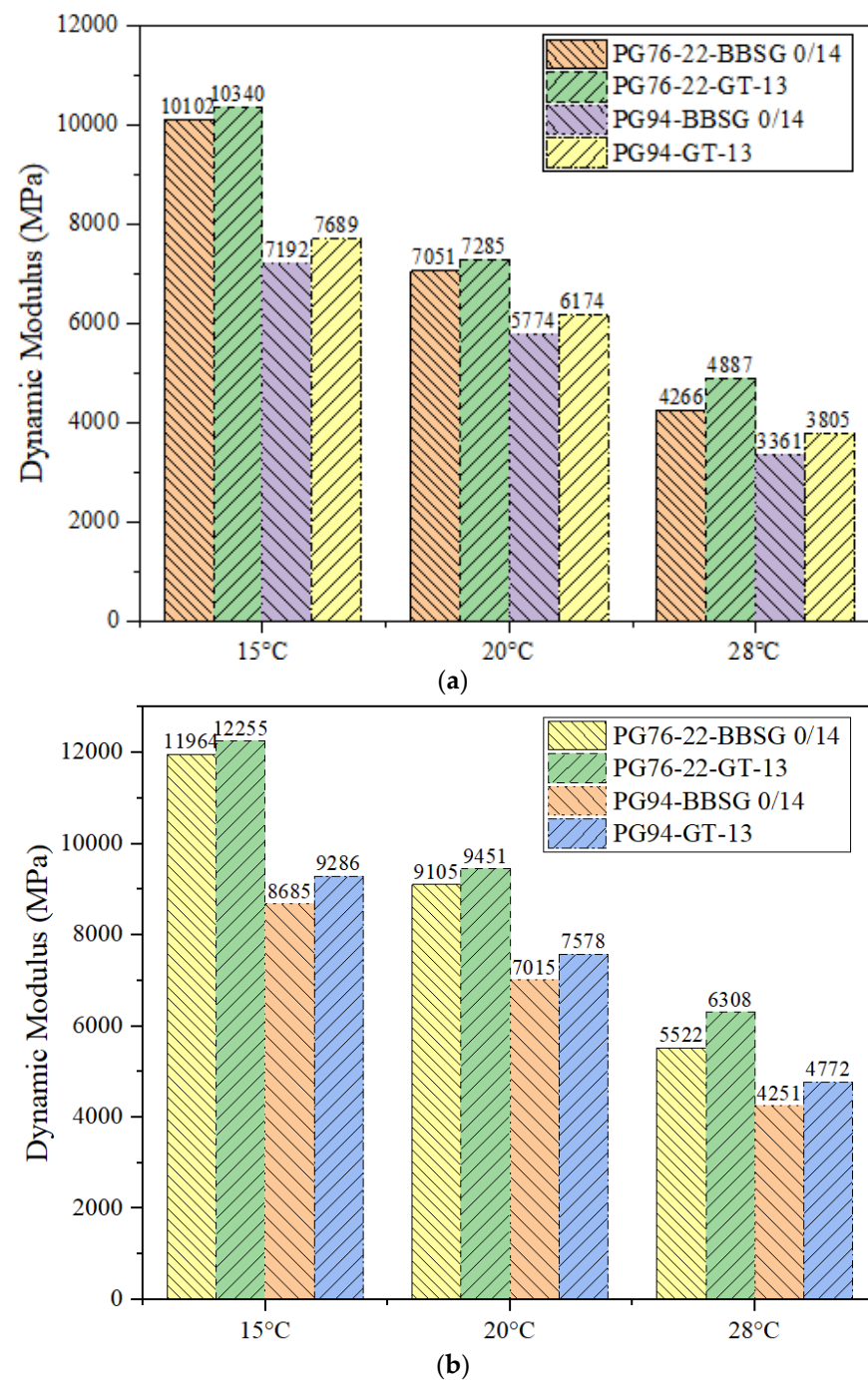


Figure 7. Dynamic modulus: (a) at 10 Hz; (b) at 25 Hz.

A two-way ANOVA considering binder type and mixture type as factors, followed by post hoc comparisons, was performed to statistically evaluate the effects of these variables on fatigue performance. The results showed that, at the 0.05 significance level, both binder type and mixture type had statistically significant effects on the overall mean fatigue life, with p -values of 0.01066 and 3.99×10^{-109} , respectively. Furthermore, the effect sizes (η^2) were calculated as 0.63 for binder type and 0.99 for mixture type, both exceeding the threshold of 0.14 for a large effect size. These results indicate that binder type and mixture type explain approximately 63% and 99% of the variance in fatigue life, respectively. The statistical analysis confirms that both binder type and mixture type significantly affect fatigue performance, with mixture type exhibiting a substantially greater influence than binder type. In addition, the CV values ranged from 0.06% to 9.97% for BBSG 0/14

mixtures and from 0.05% to 2.19% for GT-13 mixtures, indicating limited dispersion among the measured values.

Table 10. Fatigue life test results.

Binder Type	Strain Levels ($\mu\epsilon$)	Temp. ($^{\circ}\text{C}$)	BBSG 0/14 N_f (10^6 Cycles)	Flexural Stiffness (S_0) (10^4 MPa)	GT-13 N_f (10^6 Cycles)	Flexural Stiffness (S_0) (10^4 MPa)
PG94	300	10	3.19 ± 0.0018	1.20 ± 0.013	3.22 ± 0.0016	1.48 ± 0.012
	400	15	3.05 ± 0.0020	0.90 ± 0.010	3.16 ± 0.0017	1.06 ± 0.014
	500	20	0.91 ± 0.0021	0.61 ± 0.009	1.90 ± 0.0019	0.77 ± 0.010
		28	0.40 ± 0.0030	0.26 ± 0.010	1.79 ± 0.0028	0.24 ± 0.010
	800	28	0.12 ± 0.0043	0.24 ± 0.011	0.48 ± 0.0030	0.22 ± 0.009
	1000	28	0.07 ± 0.0068	0.22 ± 0.008	0.26 ± 0.0056	0.22 ± 0.010
PG76-22	300	10	2.33 ± 0.0015	1.57 ± 0.013	2.51 ± 0.0013	1.65 ± 0.016
	400	15	2.23 ± 0.0015	1.17 ± 0.001	2.43 ± 0.0014	1.41 ± 0.013
	500	20	0.71 ± 0.0016	0.80 ± 0.008	1.56 ± 0.0016	1.02 ± 0.011
		28	0.29 ± 0.0022	0.34 ± 0.010	1.29 ± 0.0021	0.31 ± 0.008
	800	28	0.09 ± 0.0031	0.32 ± 0.010	0.35 ± 0.0022	0.29 ± 0.008
	1000	28	0.05 ± 0.0049	0.28 ± 0.009	0.18 ± 0.0041	0.29 ± 0.008

3.2.1. Fatigue Life–Strain Relationship

In this experimental study, fatigue failure was defined as the point at which the stiffness modulus decreased to 50% of its initial value (N_{f50}), which is consistent with the criterion adopted in AASHTO T321-07 [39] and the French fatigue design approach implemented in Alizé-LCPC [42]. The corresponding number of load cycles was considered as fatigue life. A generalized linear regression model was fitted to the data presented in Table 10 at 28 $^{\circ}\text{C}$, which was adopted as the equivalent pavement design temperature for the Republic of the Congo, to characterize the fatigue performance of the asphalt mixtures under this failure criterion. As illustrated in Figure 8, the fatigue curves for the BBSG 0/14 and GT-13 mixtures are described by Equations (8) and (9) for PG94, and by Equations (10) and (11) for PG76-22, respectively. The perfect fitting performance ($R^2 = 1$) obtained from Equations (8)–(11) may be attributed to the limited number of experimental data points. The fatigue models were established using three strain levels (500, 800, and 1000 $\mu\epsilon$) at 28 $^{\circ}\text{C}$, with three fitting parameters. Therefore, the regression had zero degrees of freedom, leading inevitably to an R^2 value of 1.0. These relationships should be regarded as empirical models applicable within the range investigated.

$$N_{f1} = 8.100 \times 10^{12} \left(\frac{1}{\epsilon_t} \right)^{2.587} \left(\frac{1}{S_0} \right)^{0.095} (R^2 = 1) \quad (8)$$

$$N_{f2} = 9.594 \times 10^{14} \left(\frac{1}{\epsilon_t} \right)^{2.836} \left(\frac{1}{S_0} \right)^{0.320} (R^2 = 1) \quad (9)$$

$$N_{f3} = 6.096 \times 10^{12} \left(\frac{1}{\epsilon_t} \right)^{2.587} \left(\frac{1}{S_0} \right)^{0.096} (R^2 = 1) \quad (10)$$

$$N_{f4} = 9.016 \times 10^{13} \left(\frac{1}{\epsilon_t} \right)^{2.810} \left(\frac{1}{S_0} \right)^{0.076} (R^2 = 1) \quad (11)$$

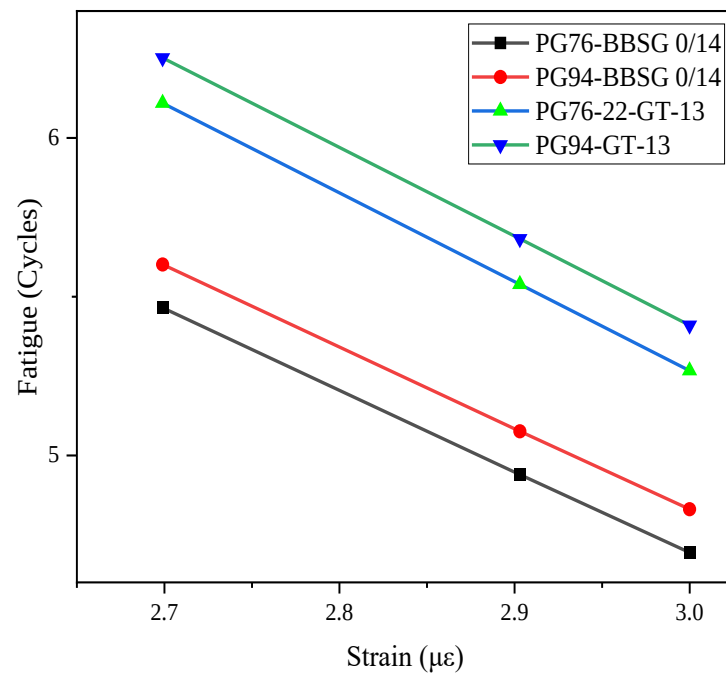


Figure 8. Fatigue resistance curves.

3.2.2. Predictive Model for Constant-Strain Fatigue Life of High-Performance Asphalt Mixtures

In the study conducted by the Road Engineering Research Institute of South China University of Technology, 2011 [43] on “Fatigue Cracking of Asphalt Layers”, a multi-variable regression analysis was performed on 618 sets of domestic and international strain-controlled four-point bending fatigue test results. Based on these strain-controlled fatigue test outcomes, a laboratory fatigue life prediction model for asphalt mixtures was developed, as illustrated in Equation (12). The fatigue life (N_f) of asphalt mixtures under laboratory constant-strain tests was expressed as a function of applied tensile strain (ϵ), initial flexural stiffness modulus (S_0), and asphalt voids filled percentage (VFA). This generalized laboratory fatigue model was originally developed based on neat (unmodified) binders and the fatigue life–strain term calibrated under standard laboratory conditions (15 °C and 10 Hz).

$$N_f = 1.509 \times 10^{16} \left(\frac{1}{\epsilon_t} \right)^{3.973} \left(\frac{1}{S_0} \right)^{1.589} (VFA)^{2.720} \quad (12)$$

High-performance modified asphalts were employed in this study. The original model established using conventional bitumen under standard conditions may lead to significant bias when applied to tropical climates, highlighting the need for binder-, temperature-, and frequency-adapted predictive formulations. Based on four-point bending fatigue tests conducted at 28 °C and a loading frequency of 25 Hz, fatigue models were developed for BBSG 0/14 and GT-13 asphalt mixtures produced with PG76 and PG94 modified binders. The VFA values remained nearly constant within each mixture type, with a value of 72% for the BBSG 0/14 mixtures regardless of binder grade, and values of 75.9% and 76.0% for the GT-13 mixtures prepared with PG76-22 and PG94, respectively. Due to the limited variation in VFA within the experimental dataset, the VFA exponent could not be independently recalibrated without introducing additional uncertainty or overfitting. Therefore, the VFA exponent (2.720) from the reference fatigue model was retained to preserve the original model structure, while the coefficients associated with tensile strain and initial flexural stiffness were recalibrated based on the fatigue test results obtained in

this study. To evaluate the influence of the adopted VFA exponent on fatigue predictions, a sensitivity analysis was conducted by varying the VFA exponent by $\pm 20\%$ (i.e., from 2.176 to 3.264). The results showed that the predicted fatigue life varied by approximately 13–19% within the investigated VFA range (72–76%), indicating that the fixed VFA exponent had a limited influence on the fatigue prediction results. By substituting Equations (8)–(11) into Equation (12), the recalibrated fatigue models for high-performance asphalt mixtures were established. The resulting models, presented in Equations (13)–(16), correspond to mixtures incorporating PG94 high-viscosity and high-elasticity modified binder and PG76-22 SBS-modified binder, respectively.

$$N_{f \text{ PG94-BBSG } 0/14} = 1.980 \times 10^{13} \left(\frac{1}{\varepsilon_t} \right)^{2.587} \left(\frac{1}{S_0} \right)^{0.096} (VFA)^{2.720} \quad (13)$$

$$N_{f \text{ PG94-GT-13}} = 2.024 \times 10^{15} \left(\frac{1}{\varepsilon_t} \right)^{2.836} \left(\frac{1}{S_0} \right)^{0.320} (VFA)^{2.720} \quad (14)$$

$$N_{f \text{ PG76-22-BBSG } 0/14} = 1.490 \times 10^{13} \left(\frac{1}{\varepsilon_t} \right)^{2.587} \left(\frac{1}{S_0} \right)^{0.096} (VFA)^{2.720} \quad (15)$$

$$N_{f \text{ PG76-22-GT-13}} = 1.909 \times 10^{14} \left(\frac{1}{\varepsilon_t} \right)^{2.810} \left(\frac{1}{S_0} \right)^{0.076} (VFA)^{2.720} \quad (16)$$

Donald Christensen established a relationship between the dynamic compressive modulus (E) and the dynamic flexural stiffness modulus (S_0) of asphalt mixtures, as shown in Equation (18) (Road Engineering Research Institute of South China University of Technology, 2011) [43].

$$|S_0| = 0.66|E|^{0.994} \quad (17)$$

When the dynamic modulus ranges from 3000 MPa to 15,000 MPa, the term $\frac{|E|^{0.994}}{E}$ is approximately equal to 0.95. Accordingly, the relationship between the dynamic compressive modulus and the dynamic flexural modulus of asphalt mixtures can be considered to remain at approximately a factor of 1.6. That is, the dynamic compressive modulus is about 1.6 times the dynamic flexural modulus under the same temperature and loading frequency conditions. Similarly, Adhikari et al. (2009) [44], based on experimental comparisons between the dynamic compressive modulus and dynamic flexural modulus of asphalt mixtures, reported that the ratio between the two is approximately 1.3. The Road Engineering Institute of South China University of Technology (2011) [43] experimentally validated the relationship between the dynamic compressive modulus and the dynamic flexural modulus. A dataset of 31 paired measurements was used to assess Equation (17). The results showed an average error of 16.8% between the predicted and measured dynamic flexural stiffness modulus, while the mean ratio between the measured dynamic compressive and flexural moduli was approximately 1.6. Although deviations exist at the individual data level, Equation (18) adequately represents the overall relationship between the two moduli. It should be noted that the conversion factor of 1.6 is an empirical relationship from previous experimental studies and may introduce uncertainty when applied to high-performance asphalt mixtures. However, this relationship was adopted in this study for model validation purposes based on previous experimental validation. Accordingly, Equation (17) was adopted to approximate the conversion between the dynamic compressive modulus and the dynamic flexural modulus. Substituting this relationship into Equations (13)–(16)

yields predictive models expressed in terms of dynamic compressive modulus, as given in Equations (18)–(21):

$$N_{f \text{ PG94-BBSG } 0/14} = 2.061 \times 10^{13} \left(\frac{1}{\varepsilon_t} \right)^{2.587} \left(\frac{1}{E} \right)^{0.0954} (VFA)^{2.720} \quad (18)$$

$$N_{f \text{ PG94-GT-13}} = 2.312 \times 10^{15} \left(\frac{1}{\varepsilon_t} \right)^{2.836} \left(\frac{1}{E} \right)^{0.318} (VFA)^{2.720} \quad (19)$$

$$N_{f \text{ PG76-22-BBSG } 0/14} = 1.551 \times 10^{13} \left(\frac{1}{\varepsilon_t} \right)^{2.587} \left(\frac{1}{E} \right)^{0.0954} (VFA)^{2.720} \quad (20)$$

$$N_{f \text{ PG76-22-GT-13}} = 1.970 \times 10^{14} \left(\frac{1}{\varepsilon_t} \right)^{2.810} \left(\frac{1}{E} \right)^{0.076} (VFA)^{2.720} \quad (21)$$

3.2.3. Validation of the Predictive Model

The verification procedure for the proposed constant-strain fatigue life prediction model comprises the following sequential steps:

- (1) Experimental determination of the measured fatigue life (N_{fM}) under four-point bending test conditions;
- (2) Estimation of the predicted fatigue life (N_{fP}) using the constant-strain fatigue life prediction model for high-performance asphalt mixtures (Equations (18)–(21));
- (3) Evaluation of the model's predictive accuracy by comparing N_{fE} with N_{fP} , with a relative deviation (RD) threshold of 20% adopted as the criterion for acceptable precision.

The model was validated using experimental data obtained from the laboratory tests described above. The validation results are summarized in Table 11. For the four mixture types investigated, the fatigue lives predicted by Equations (18)–(21) showed only slight deviations from the measured values, with a maximum relative deviation of 10.3%, which remains within the 20% tolerance threshold. Furthermore, a strong correlation was observed between the N_{fP} and N_{fM} , yielding a coefficient of determination (R^2) of approximately 99.99%. This finding indicates that the refined model offers a notable enhancement in prediction accuracy relative to the original model.

Table 11. Validation results.

Mix Type		Strain Levels ($\mu\varepsilon$)	Predicted Value N_{fP}	Measured Value N_{fM}	Relative Deviation RD (%)
PG94	BBSG 0/14	500	396,093	399,400	0.83
		800	117,419	119,115	1.42
		1000	65,922	67,675	2.59
	GT-13	500	1,643,218	1,788,525	8.12
		800	433,322	481,305	9.97
		1000	230,130	256,800	10.39
PG76-22	BBSG 0/14	500	290,620	291,562	0.32
		800	86,152	86,950	0.92
		1000	48,368	49,400	2.09
	GT-13	500	1,246,885	1,287,738	3.17
		800	332,851	345,650	3.70
		1000	177,800	184,900	3.84

The applicability of the proposed fatigue model should be considered within the climatic conditions investigated in this study. The model was developed and validated under hot and humid tropical conditions representative of the Republic of the Congo, using an equivalent design temperature of 28 °C. Therefore, its application to other climatic regions with different temperature conditions requires further validation and possible recalibration of the temperature-related parameters.

3.3. Case Study

3.3.1. Structure and Parameters of Pavement

For the current study, the RN2 national road (Figure 9), serving as critical infrastructure for both domestic freight logistics and regional economic integration within the Congo's infrastructure network, was considered. This strategic corridor connects the country's key economic and political centers across its southern and northern regions, including Brazzaville, the political capital and administrative heart of the country, and Ouessou, a secondary economic pole. The study area exhibits mean annual precipitation ranging from 1400 mm to 1700 mm, coupled with average annual temperatures fluctuating between 27.6 and 28.8 °C. Pavement structures were designed in compliance with French specifications. The pavement design life was set at 20 years for a heavy goods vehicle traffic class of T3+. Based on this traffic classification, the cumulative number of equivalent standard axle loads (NPL) over the design period was estimated at 1.6151×10^6 . According to the LCPC-SETRA pavement design approach [42] and NF P98-086 [35], a risk level of 18% was adopted for this traffic category to account for uncertainties related to traffic loading, material characteristics, and pavement performance prediction. This design approach ensures adequate pavement reliability while optimizing the overall cost over the infrastructure service life. A temperature of 28 °C was adopted as the equivalent pavement design temperature for fatigue analysis, consistent with mechanistic–empirical pavement design principles, where calculations are performed at a reference temperature representative of cumulative long-term damage. This ensures that the calculated strains and fatigue life are representative of in-service conditions, avoiding overestimation of pavement performance.

To estimate the fatigue life of the four asphalt mixtures, dynamic modulus values measured at 25 Hz and 28 °C were used as inputs in the French Alizé-LCPC pavement analysis program. These conditions were selected to be consistent with those adopted in the four-point bending fatigue tests, ensuring that the stiffness and fatigue parameters were determined under the same laboratory loading frequency and temperature conditions, thereby reducing the potential influence of differences in testing conditions on fatigue life estimation. The use of 28 °C corresponds to the equivalent design temperature adopted for pavement analysis in tropical conditions such as the Republic of the Congo. However, it should be noted that a single dynamic modulus value cannot fully represent the variations in temperature and loading frequency experienced in actual pavement structures, and the 25 Hz laboratory frequency does not directly represent the actual loading frequency associated with field traffic conditions. This standardized frequency was adopted according to the French pavement material characterization procedures and provides a consistent basis for comparing the fatigue performance of different asphalt mixtures. Alize-LCPC is one of the most used applications for analyzing flexible pavement systems in tropical countries such as the Republic of the Congo. It focuses on the elastic homogeneous isotropic multilayer theory model to predict stresses and strains in pavement structures induced by the reference standard dual wheel loaded at 130 kN, which has wheel spacing of 0.375 m, a radius of 0.125 m, and 0.662 MPa of pressure [36]. The pavement structure and material parameters are listed in Tables 12 and 13. The single variable considered in this analysis

was the material of the surface layer, i.e., the pavement structures differed exclusively in the asphalt concrete (AC) layer material. Meanwhile, the remaining layer was constant for all cases. The AC layer thickness ranged from 50 to 100 mm, with specific thicknesses of 50, 60, 70, 80, 90, and 100 mm.



Figure 9. Location of RN2. Source: cityvission.cn.

Table 12. Structure and pavement parameters.

Pavement Layer	Material Type	Thickness/mm	Modulus/MPa (28 °C)	Poisson's Ratio
Surface	AC	50–100	Vary depending on mix type	0.40
Base	GNT1	250	600	0.35
Subbase	GNT2	200	400	0.35
Subgrade	PF3	-	120	0.35

Notes: AC (asphalt concrete); GNT 1 (Grave Non Traité); PF3 (Plate-Forme).

The N_f of each experimental pavement structure were predicted in terms of the allowable number of reference loads.

Table 13. Modulus and Poisson's ratio for each layer.

Mix Type	Modulus/MPa (28 °C, 25 Hz)	Poisson's Ratio
PG76-22-BBSG 0/14	5994	0.4
PG94-BBSG 0/14	4251	0.4
PG76-22-GT-13	6308	0.4
PG94-GT-13	4772	0.4

3.3.2. Allowable Strain

According to the French design method [2,36], the maximum allowable tensile strain at the bottom of a bituminous layer (ε_{t-adm}) is calculated to be

$$\varepsilon_{t-adm} = \varepsilon_6(28\text{ °C}; 25\text{ Hz}) \times \left(\frac{NE}{10^6}\right)^b \times k_c \times k_r \times k_s \times k_\theta \quad (22)$$

Here, $\varepsilon_6(28\text{ °C}; 25\text{ Hz})$ represents the mean strain amplitude that results in the fatigue curve after 10^6 loading cycles; b represents the slope of the fatigue law of the material layer; NE is the equivalent number of reference axes; k_c and k_s are the adjustment coefficients; k_r represents the risk coefficient; and k_θ is the temperature effect coefficient on the fatigue of bituminous materials.

The allowable strain in the top surface of untreated gravels and subgrade layers (ε_{z-adm}) is calculated using Equation (23):

$$\varepsilon_{z-adm} = 12000 \times NE^{-0.222} \quad (23)$$

Fatigue cracking parameters and design constants for each asphalt concrete and granular layers are listed in Table 14.

Table 14. Material parameters.

AC Mixture	PG94-BBSG 0/14	PG76-22-BBSG 0/14	PG94-GT-13	PG76-22-GT-13
$\varepsilon_6(28\text{ °C}; 25\text{ Hz})$ ($\mu\epsilon$)	348	308	548	616
B (28 °C; 25 Hz)	−0.391	−0.391	−0.257	−0.257
E (10 °C; 25 Hz)	11,601	20,908	13,159	16,214
k_θ	1.65	1.87	1.66	1.60
k_r	0.723	0.723	0.722	0.722
k_c	1.1	1.1	1.1	1.1
k_s	1	1	1	1

3.3.3. Pavement Response Results

According to the NE and risk value, the horizontal strain at the bottom of the AC layer and vertical strain on the top surface of the subgrade were calculated using Alize-LCPC v2.3.1 software. Figure 10 shows the comparison of horizontal strain distribution. It is evident that the tensile strain in the AC layer is highest for the pavement incorporating the PG94-BBSG 0/14 mixture, followed in decreasing order by the PG94-GT-13, PG76-22-BBSG 0/14, and PG76-22-GT-13 mixtures. This is because the PG94-BBSG 0/14 mixture exhibits the lowest dynamic modulus value (Table 12), leading to the highest strain value. Furthermore, regardless of the asphalt mixture, the maximum tensile strain in the AC layer occurs at the bottom when the layer thickness is 70 mm, whereas it remains approximately the same at 50 mm.

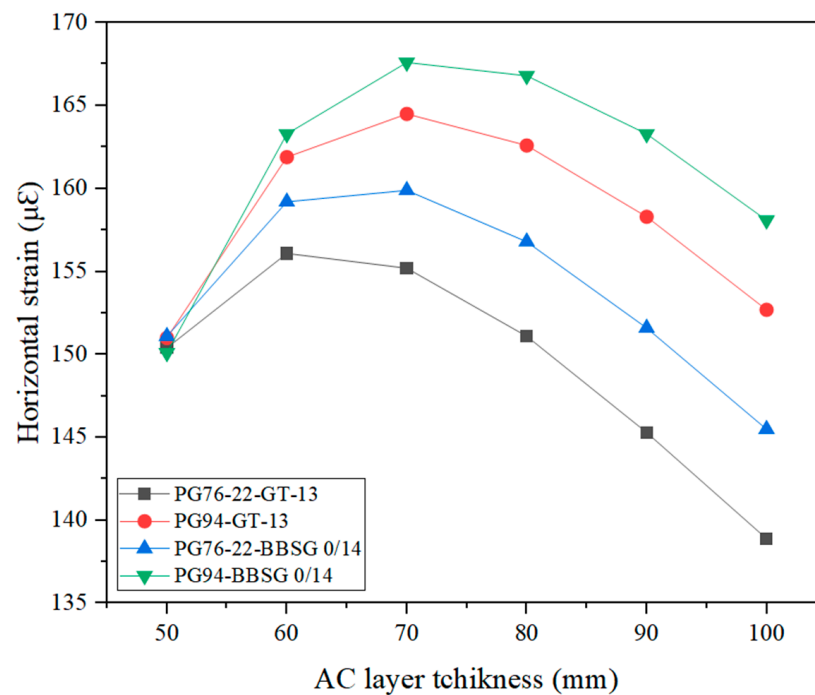


Figure 10. Comparison of horizontal strain at the bottom of the AC layer.

Moreover, the effect of asphalt layer thickness on the vertical compressive strain at the top of the subgrade is presented in Figure 11. As shown in the figure, the vertical compressive strain decreases dramatically as the asphalt thickness increases for all AC mixtures. This is because a greater asphalt thickness enhances the overall stiffness of the pavement structure, thereby reducing the vertical compressive stress and, consequently, the corresponding strain at the top of the subgrade. This indicates that the asphalt layer thickness plays a critical role in mitigating permanent deformation at the top of the subgrade, as well as rutting on the pavement surface. Among the mixtures, the pavement structure with the PG94-BBSG 0/14 mix exhibited the highest vertical strain value, followed in decreasing order by the PG94-GT-13 mix, PG76-22-BBSG 0/14, and finally the PG76-22-GT-13 mix. The PG76-22-GT-13 mix has a higher dynamic modulus (E) compared to PG94-BBSG 0/14, resulting in a lower vertical strain value at the top of the subgrade. Furthermore, among the two mixtures prepared with PG76-22, the PG76-22-GT-13 mix exhibits higher E values than the PG76-22-BBSG 0/14 mix. This difference may be attributed to the influence of aggregate gradation and binder content. These findings further confirm the importance of selecting appropriate aggregate gradation and binder type to prevent fatigue cracking and rutting distress in the pavement. Comparable findings were observed in the study of Do et al. (2025) [18].

3.3.4. Fatigue Life Prediction

Since the pavement structural response results related to the effect of asphalt thickness exhibited a similar trend, only those for the structure with a 50 mm surface course were selected for comparison (see Table 15).

Based on the results of the horizontal strain at the bottom of the AC layer, the equivalent number of reference axles to failure due to fatigue was calculated for all cases of pavement structure by using Equations (18)–(21). The predicted fatigue cracking life and associated damage for all pavement structures are shown in Table 16 and Figure 12.

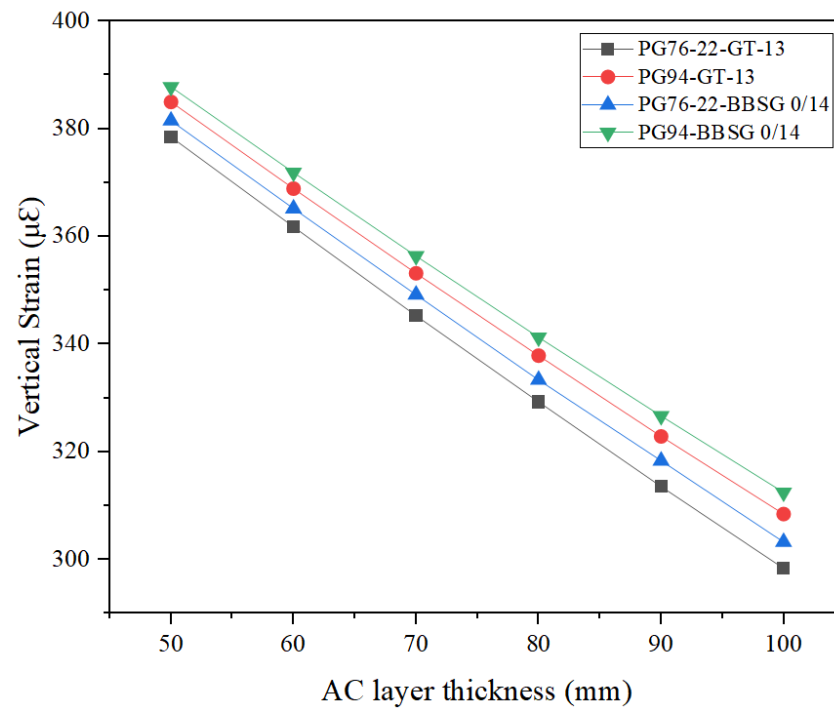


Figure 11. Comparison of vertical strain on top of the subgrade.

Table 15. Internal strain results (μϵ).

Pavement Cases	AC Layer	AC Bottom Lay.		PF 3	
		ϵ_f	ϵ_{t-adm}	ϵ_r	ϵ_{z-adm}
Structure 1	PG94-BBSG 0/14	150.1	413.7	387.8	502.3
Structure 2	PG76-22-BBSG 0/14	151.1	431.2	381.5	502.3
Structure 3	PG94-GT-13	151.0	741.3	385.0	502.3
Structure 4	PG76-22-GT-13	150.0	636.7	378.4	502.3

Table 16. Fatigue life prediction results.

Pavement Cases	Structure 1	Structure 2	Structure 3	Structure 4
N_f	8,907,094	6,423,963	49,022,498	36,738,012
NE	1,293,200	1,293,200	1,293,200	1,293,200
D	0.145	0.201	0.026	0.035
$1/D$	6.90	4.98	38.46	28.57

Table 15 reveals a significant correlation between mechanical performance and the type of asphalt and binder grade used. A clear performance hierarchy is observed: the PG94-GT-13 mix provides the greatest resistance to fatigue, followed in descending order by PG76-22-GT-13, PG94-BBSG 0/14, and finally PG76-22-BBSG 0/14.

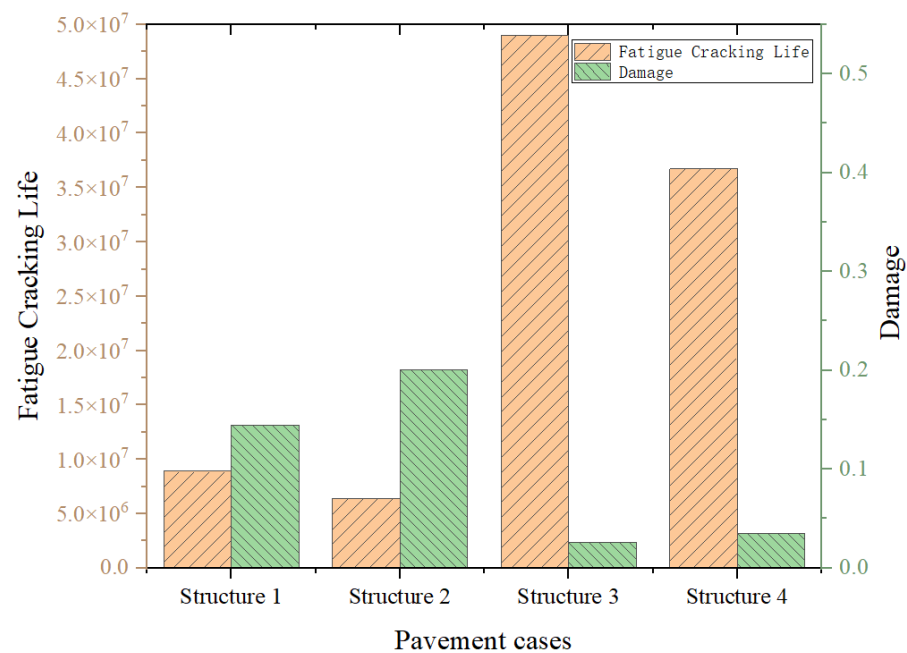


Figure 12. Predicted fatigue cracking life and associated damage for all pavement structures.

When BBSG 0/14 is used as the asphalt surface layer, the fatigue life of the pavement for Structure 1, with PG94-BBSG as the surface layer, is 1.39 times longer than that of Structure 2 with PG76-22 as the surface layer. The cumulative fatigue damage decreases from 0.201 to 0.145, with corresponding inverse values between 4.98 and 6.90. These results indicate that, when different asphalt binders are used, the pavement structure is susceptible to early failure, and fatigue cracking of the asphalt wearing course may occur within approximately 5 years. This finding is consistent with field investigations conducted in the Republic of the Congo, where some sections of the roadway exhibited surface cracking within 5 years of service. This agreement suggests that the proposed fatigue prediction model is representative and reliable.

Compared with BBSG 0/14, GT-13 demonstrates significant advantages in pavement structural performance. The fatigue life of the pavement increases substantially, reaching more than 5.74 times that of BBSG 0/14, while the cumulative fatigue damage decreases to a range of 0.201 to 0.026, corresponding to a reduction of approximately 87%. Consequently, the equivalent theoretical fatigue design life estimated for the asphalt layer exceeds 28 years for Structure 4. Furthermore, the equivalent fatigue life of Structure 3, with PG94-GT-13 as the surface layer, is 1.35 times higher than that of PG76-22-GT-13 (Structure 4). This enhanced performance is mainly related to the internal structural characteristics of GT-13. Compared with the more continuous aggregate structure of BBSG 0/14, GT-13 exhibits a stone-on-stone skeleton, where coarse aggregates form a stable load-bearing framework through direct contact. This structure improves stress distribution within the mixture and contributes to lower tensile strain levels in the pavement structure. Moreover, the appropriate binder content and volumetric properties of GT-13, including VMA, provide favorable conditions for binder accommodation and aggregate coating, contributing to mixture durability. The thermodynamic adhesion energy between binder and aggregate, which may affect interfacial bonding, was not evaluated in this study and requires further investigation. These results are consistent with those of previous studies conducted on asphalt mixtures incorporating modified binders [43–45]. However, the estimated fatigue lives should be interpreted as equivalent theoretical design values obtained from laboratory characterization and mechanistic analysis, assuming consistent material properties, construction quality, and environmental conditions. Overall, the combination of a stable

aggregate skeleton in GT-13 and high-performance binders provides a promising solution for improving fatigue resistance and rutting stability in high-traffic pavement applications under tropical conditions, such as those encountered in the Republic of the Congo and other regions with similar climatic and traffic conditions. It should be noted that the BBSG 0/14 mixtures were evaluated with modified binders only; thus, the contribution of binder modification relative to conventional solutions remains to be quantified.

3.4. Composite-Index

The 2026 cost breakdown from the Congolese Road Maintenance Authority (DGER) is presented in Table 17. The techno-economic evaluation based on the composite indices highlights the combined influence of binder grade and mixture type on the overall cost-effectiveness of pavement structures. The results presented in Table 18 and Figure 13 show that Structure 2 with PG76-22-BBSG 0/14 as the surface layer exhibits higher composite indices ($CLR = 0.055 \text{ XAF/m}^2/\text{Nf}$ and $MEI = 0.0028 \text{ XAF/m/Nf}$), indicating lower economic efficiency compared with the other evaluated structures. In contrast, the PG94-BBSG 0/14 mixture achieves a CLR of $0.0523 \text{ XAF/m}^2/\text{Nf}$ and an MEI of 0.0026 XAF/m/Nf , showing improved indicators compared with Structure 2. The GT-13 mixtures exhibit lower values, with PG76-22-GT-13 recording a CLR of $0.0098 \text{ XAF/m}^2/\text{Nf}$ and an MEI of 0.0005 XAF/m/Nf , while PG94-GT-13 achieves the lowest values ($CLR = 0.0096 \text{ XAF/m}^2/\text{Nf}$ and $MEI = 0.0004 \text{ XAF/m/Nf}$). These results indicate differences in the balance between cost, durability, and material efficiency among the evaluated mixtures. A sensitivity analysis was subsequently conducted by varying the PG94/PG76-22 price ratio from 1.5 to 3.0. The results indicate that GT-13 mixtures maintain lower CLR and MEI values than BBSG mixtures throughout the range investigated. At the current price ratio (~ 2.0), PG94-GT-13 presents the lowest indicators; however, this difference decreases as the PG94 price increases, and PG76-22-GT-13 becomes more favorable when the ratio exceeds approximately 2.1. The ranking between GT-13 and BBSG mixtures remains unchanged, indicating that binder price variations have a limited effect on the overall economic ranking.

Table 17. Cost breakdown of materials in 2026.

S.N°	Material Type	Unit	Cost (XAF)
1	PG76-22	Ton	737,100
2	PG94	Ton	1,458,360
3	PG76-22-BBSG 0/14 (5 cm)	m ²	353,662
4	PG94-BBSG 0/14 (5 cm)	m ²	469,026
5	PG76-22-GT-13 (5 cm)	m ²	357,914
6	PG94-GT-13 (5 cm)	m ²	473,278
7	GNT 0/31 (25 cm)	m ²	110,875
8	GNT 0/25 (20 cm)	m ²	78,272

Table 18. Composite indices' results.

Asphalt Mixture	Life (N _f)	Coast ($\frac{\text{XAF}}{\text{m}^2}$)	Hm (m)	CLR ($\frac{\text{XAF}}{\text{m}^2 \times \text{N}_f}$)	MEI ($\frac{\text{XAF}}{\text{m} \times \text{N}_f}$)
PG76-22-BBSG 0/14	6,423,963	353,662	0.05	0.055	0.0028
PG94-BBSG 0/14	8,907,094	469,026	0.05	0.053	0.0026
PG76-22-GT-13	36,738,012	357,914	0.05	0.0098	0.0005
PG94-GT-13	49,022,498	473,278	0.05	0.0096	0.0004

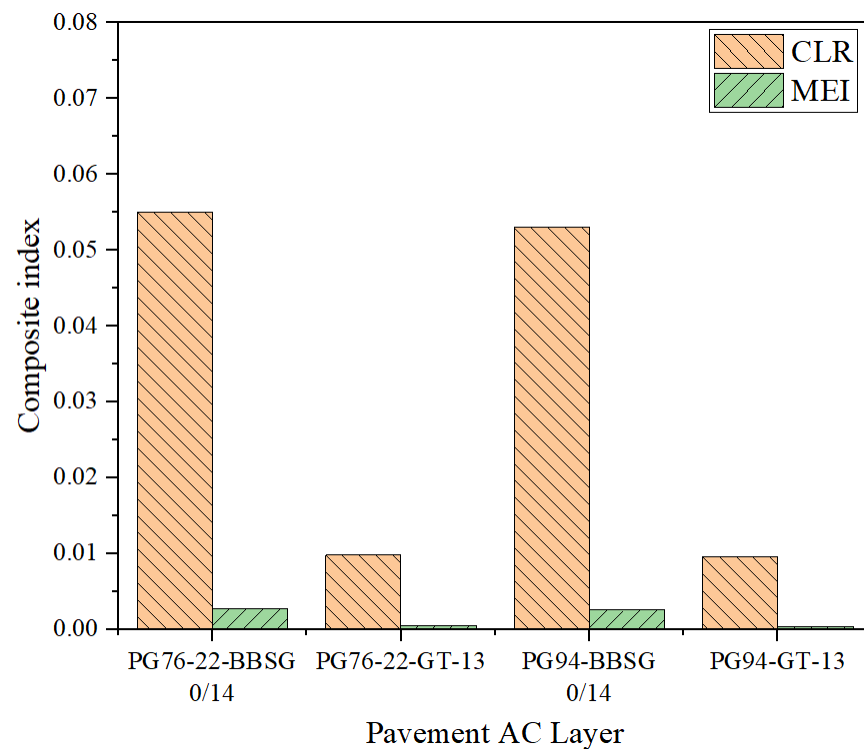


Figure 13. Composite indices.

Therefore, from a long-term performance and economic sustainability perspective, GT-13 demonstrates enhanced durability and cost-effectiveness as a surface course material for the Republic of the Congo.

The symbols used in this article and their meanings are summarized in Table 19.

Table 19. Symbol index.

Symbol	Meaning
VCA_{DRC}	Percentage voids in the coarse aggregate skeleton
BSG	Specimens' bulk specific gravity
AV	Air voids
VMA	Voids in mineral aggregate
VFA	Voids filled with asphalt
E^*	Dynamic modulus for frequency
ϵ^*	Average strain magnitude
S_0	Dynamic flexural stiffness modulus
Nf	The fatigue life
D	Fatigue damage
N_{PL}	Heavy goods vehicles
NE	Reference axle loads
CAM	The average traffic aggressiveness coefficient
CLR	Cost-Life Ratio
MEI	Material Efficiency Index

4. Conclusions

This research highlights the decisive influence of binder grade and aggregate structure on the performance of asphalt mixtures under tropical conditions. Conventional BBSG 0/14, though widely used, shows limited durability in the tropical climate, whereas the denser GT-13 mixture demonstrates higher mechanical behavior and economic sustainability.

Compared to mixtures made with PG76-22, the mixture with PG94 had the lowest stiffness value across all testing temperatures and frequencies. At each test temperature and frequency, GT-13 had the highest elastic modulus value among the two asphalt binders compared to BBSG 0/14. When formulated with PG76-22 binder, GT-13 exhibited a 2.4%, 3.3%, and 14.6% higher dynamic modulus at 15 °C, 20 °C, and 28 °C, respectively. With PG94, the corresponding increases were 6.91%, 6.93%, and 13.21% at the same temperatures, respectively. At high temperatures, the dynamic modulus of the GT-13 mixture exceeded that of the BBSG 0/14 mixture by approximately 12% to 14%, depending on frequency and binder type, illustrating the enhanced efficacy of the GT-13 mixture in reducing tensile strains at the bottom of the asphalt layer.

High-performance binder significantly improves fatigue resistance, while the GT-13 gradation enhances internal cohesion and load distribution. Together, these properties produce a synergistic effect, extending the pavement life by more than 5.7 times compared with conventional BBSG 0/14. The equivalent theoretical design life of the asphalt wearing course may occur within approximately 6 years for the conventional BBSG 0/14, whereas it is extended to more than 28 years with GT-13. The result of BBSG 0/14 is consistent with field investigations conducted in the Republic of the Congo, where some sections of the roadway exhibited surface cracking within 5 years of service. This agreement suggests that the proposed fatigue prediction model is representative and reliable.

Techno-economic indicators further confirm that PG94-GT-13 offers the optimal balance among initial cost, performance, and maintenance frequency, making it the optimal design choice for major highways' and freight corridors' surface course material for the Republic of the Congo. Future work should focus on validating these laboratory findings through field studies, developing localized specifications for modified binders adapted to African tropical climates, and evaluating the proposed model using additional fatigue test results obtained under a wider range of strain levels and temperatures.

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