Deterioration Analysis of Pavement Structures Incorporating Polymer-Modified Asphalt

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ABSTRACT

It is well-established that structural pavement deterioration is largely influenced by the frequency and magnitude of wheel loads. Furthermore, Polymer-Modified Asphalt (PMA) has gained a widespread application in pavement engineering, as the incorporation of polymers enhances the mechanical properties and improves the overall performance of the pavement, particularly with regard to fatigue resistance. In this study, an experimental program was conducted, comprising the Beam Wheel Tracker Fatigue Test (BWTFT), the Indirect Tensile Stiffness Modulus (ITSM) test, and the Indirect Tensile Fatigue Test (ITFT), on three types of asphalt mixtures: one conventional and two polymer-modified asphalt mixtures, under various conditions. Three distinct pavement structure scenarios were assumed to perform a deterioration analysis. Subsequently, an iterative approach was developed utilizing the average stiffness reduction and stiffness modulus data obtained from the laboratory results. The findings indicated that this method allows for a more accurate simulation of pavement behavior, confirming that strain levels fluctuate throughout the lifespan of the pavement. Furthermore, the study concluded that the use of PMA provides significantly greater benefits when a deterioration analysis is conducted, compared to traditional approaches.

Keywords-deterioration analysis; pavement structures; asphalt mixtures; polymer-modified asphalt

I. INTRODUCTION

In the field of pavement engineering, the fatigue life of bituminous materials has traditionally been defined as the number of load applications the materials can withstand before reaching failure at a specific stress or strain level [1-3]. Pavement design commonly utilizes the strain criterion, with the maximum strain amplitude established through laboratory testing to determine the fatigue line. Conventional calculations, however, do not account for material deterioration, assuming a constant strain throughout the pavement's life. This assumption fails to reflect the actual conditions under traffic, where neither strain nor stress remain constant [4-5]. During the fatigue process, the effective stiffness of the material decreases due to the gradually diminishing resistance of the asphalt mixture under repeated loading.

Over the past several years, PMA has become the industry standard for constructing high-performance pavements worldwide. Pavements incorporating polymer-modification demonstrate enhanced resistance to permanent deformation, thermal cracking, fatigue damage, moisture-induced stripping, and temperature sensitivity [6-8]. Although numerous laboratory and field investigations have validated the improved performance of PMA compared to conventional hot mix asphalt, there has been a lack of extensive research to comprehend the detailed mechanical behavior of PMA and provide reliable pavement design guidance incorporating PMA based on relevant specifications [9-14].

A key challenge in pavement design is establishing a meaningful relationship between laboratory and field performance. Even though laboratory tests often employ sinusoidal loading and fixed strain or stress during fatigue testing, real-world pavement loading is randomly distributed. Actual pavement life is influenced by rest periods between load applications, the time for crack propagation through the asphalt layer, and lateral distribution of wheel loads on the surface [15]. To account for these effects, a correction factor, known as shift factor, is typically applied when transitioning from laboratory to field conditions. However, estimating an accurate shift factor is complex, as it depends on the fatigue test type, loading mode, test temperature, and mixture type [16].

The purpose of this paper is to examine pavement structures that incorporate PMA materials, considering that strains vary throughout the pavement's lifespan. To achieve this, three case scenarios were developed for different materials, two which use either a conventional or a PMA mixture as the base course, and one which utilizes another PMA mixture as the surface course. An iterative method based on fatigue data was developed and applied to conduct a deterioration analysis of the pavements using the KENLAYER program. In this analysis, the average stiffness reduction data from the BWTFT were employed to simulate the degradation of the mixtures under repeated loads [17]. The results were then compared with those obtained using a traditional approach. This paper investigates pavement structures that incorporate PMA materials, accounting for the varying strains throughout the pavement's lifetime.

II. EXPERIMENTAL PROGRAM

A. *Tested Materials*

This study examined three different asphalt mixtures to assess pavement structures incorporating PMA materials: a 20 mm Dense Bitumen Macadam (DBM50), a 14 mm Enrobé à Module Élevé 2 (EME2), and a 10 mm Stone Mastic Asphalt (SMA). For the DBM50 mixture, a standard 20 mm continuously graded aggregate was utilized. The binder grade used was 40/60 bitumen, commonly employed in pavement construction. The binder content was 4.7% by mass, conforming to BS-EN 4987 [18]. The aggregate was limestone, sourced from Buxton, United Kingdom, with a nominal maximum particle size of 20 mm. The second mixture, the standard French 14 mm EME2, had a gradation closely resembling that of DBM50, as both are intended for the same structural layer. This mixture utilized a hard SBS polymer-modified binder, 21 pen, with a binder content of 5.5% by mass, as proposed by BS-EN 13108 [19]. Similarly to the DBM50, the aggregate was limestone from Buxton, United Kingdom, with a nominal maximum particle size of 14 mm. The third mixture chosen was a standard 10 mm SMA for the surface course, using a soft SBS polymer-modified binder, 65 pen, at a content of 6.5%, along with granite aggregate, designed in accordance with BS-EN 13108 [20]. The design particle size distribution used for all DBM50, EME2 and SMA mixtures is presented in Figure 1.

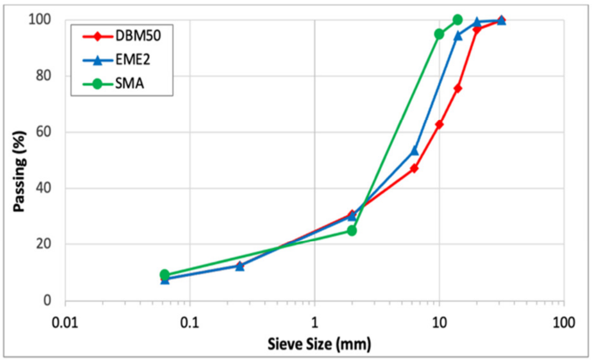

Fig. 1. Design particle size distribution for the three mixtures under study.

B. *Laboratory Testing*

1) *BWTFT*

This study utilized the BWTFT to assess mixtures at a temperature of 20 °C. The experiments were conducted in a temperature-controlled environment. A cam system oscillated the specimen relative to the wheel at a frequency of 0.4 Hz, as depicted in Figure 2. Two strain gauges were attached to the bottom of each vertical face and connected to a computer system equipped with national instrument equipment. Furthermore, a camera was employed to directly monitor crack formation, with the faces painted white to enhance visibility.

The asphalt beam specimens investigated in this study were 306 mm long, 50 mm wide, and 35 mm tall. The test specimen was centrally positioned within a steel mould and supported on a rubber pad, which provided an elastic foundation for the asphalt beam. Additionally, two steel end restraints were used to secure both ends of the test beam, limiting vertical displacement and emulating the performance of a continuous beam system.

The BWTFT procedure involved applying a wheel load to the mobile asphalt beam while recording the tensile strains along its sides. A range of load levels, from 0.76 to 1.50 kN, were utilized. Prior to commencing the experiment, the beams equipped with attached gauges were conditioned at the test temperature for a minimum duration of four hours. The trials persisted until a visible crack propagated through the entire thickness of the beam.

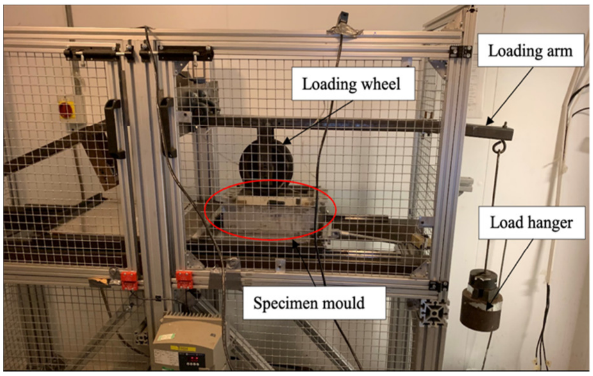

Fig. 2. Schematic of the BWTFT.

2) *ITSM Test*

The ITSM is a straightforward, non-destructive, expedient, and cost-effective technique for evaluating the stiffness modulus of asphalt composites. It can be readily implemented using the Nottingham Asphalt Tester (NAT) for assessing the stiffness characteristics of asphalt mixtures. The cylindrical specimens employed for both the ITSM test and ITFT were extracted from slabs fabricated in the laboratory. The final dimensions of the test specimens were 100 mm in diameter and 40 mm in height. Prior to testing, all cylindrical specimens were conditioned for a minimum of four hours at 20 °C. The ITSM test was utilized to evaluate the stiffness modulus of DBM50, EME2, and SMA mixture specimens [21]. A NAT machine was employed for these tests. A pulsating load was applied centrally between the upper and lower platens, and the peak transient deformation along the horizontal diameter was measured, as shown in Figure 3. The target horizontal deformation was set at 5 ± 2 μm, with an assumed Poisson's ratio of 0.35 and a target rise time of 124 ± 4 ms.

3) *ITFT*

The ITFT is utilized to assess the stiffness and fatigue behavior of asphalt mixtures. This test method is favored due to its straightforward nature and compatibility with cylindrical specimens, which can be either laboratory-produced or extracted from existing asphalt pavement structures [22-24].

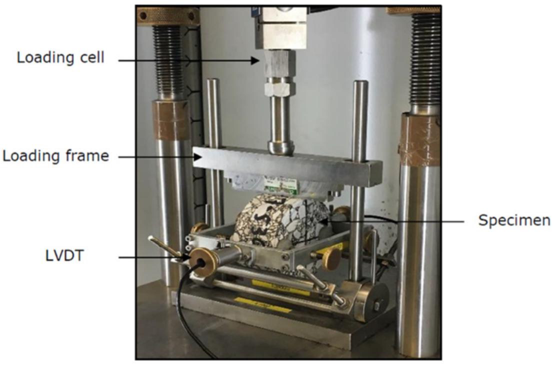

Fig. 3. Schematic of the ITSM test.

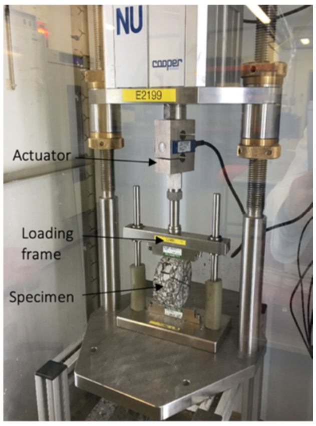

Fig. 4. Schematic of the ITFT.

Prior to the ITFT, the Indirect Tensile Stiffness Test (ITST) was performed to assess the stiffness of the specimens at different stress levels. These same specimens were then subjected to the ITFT to determine the number of load applications required to cause failure at the stress level used in the ITST. Additionally, the specimens were conditioned in a NAT cabinet for at least four hours at the/a testing temperature before commencing the ITFT.

In this study, the ITFT was conducted in a stress-controlled manner within the NAT at a temperature of 20 °C. The test utilized a repeated constant loading time of 124 ± 4 ms and a pulse repetition time of 1.5 ± 0.1 s [25]. The specimen was considered to have failed when the total vertical deformation reached 10 mm. A schematic depiction of the ITFT is displayed in Figure 4.

III. PAVEMENT STRUCTURE SCENARIOS AND MATERIAL PROPERTIES

The KENLAYER program is a pavement design software deploying a multi-layer linear elastic analysis [26]. It exhibits greater versatility compared to the Bitumen Stress Analysis in Roads (BISAR) as its failure criteria can be modified. The software's failure criteria are adjusted by altering the fatigue cracking and permanent deformation laws [4]. The KENLAYER program outputs encompass strains, stresses, and deflections within pavement layers, and the final design life is determined based on the allowable number of load applications defined by the failure laws. This widely available computer software is regarded as a valuable tool for pavement design, and was therefore employed for pavement analysis and design in this study. The analysis utilized a standard single wheel load of 40 kN, 566 kPa contact tire pressure, and a contact area radius of 150 mm. The KENLAYER program was used to analyze the deterioration within the DBM50 or EME2 binder and base, as well as the SMA surface layer. Diagrams depicting pavement structures 1, 2, and 3 are presented in Figures 5 and 6. To predict the strain evolution in the pavements, the base layer with DBM50 or EME2 and the surface course with SMA were divided into 4 sub-layers in KENLAYER.

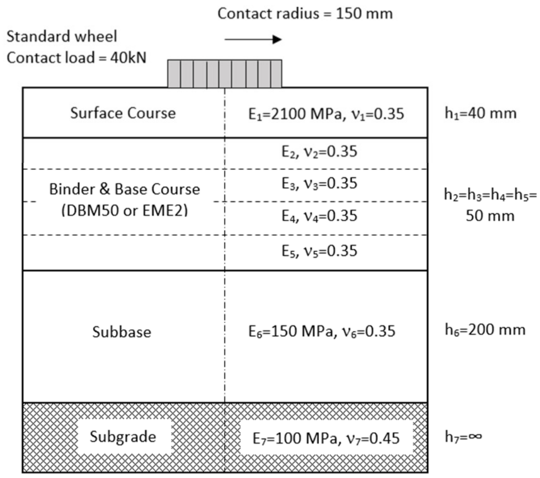

Fig. 5. Pavement 1 and 2 for the binder and base analysis.

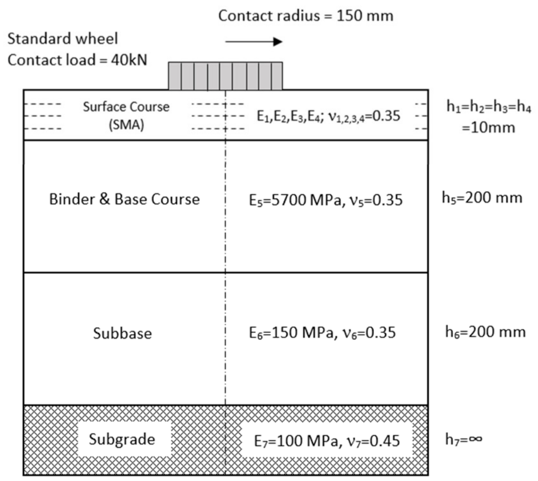

Fig. 6. Pavement 3 for the surface analysis.

To conduct a KENLAYER analysis, stiffness and Poisson's ratio values are required. The 15th percentile of laboratory stiffness measurements can be used as a conservative approach for design purposes [4]. Accordingly, the stiffness values of DMB50, EME2, and SMA were obtained from the ITSM test at a design temperature of 20 °C, as shown in Table I [27]. A Poisson's ratio of 0.35 is assumed for asphalt and subbase layers, while a value of 0.45 is used for the subgrade, as indicated in the previous Figures. Additionally, the thickness and stiffness of the remaining layers used in the KENLAYER program are assumed as depicted in Figures 5 and 6.

TABLE I. STIFFNESS VALUES OF DBM50, EME2 AND SMA AT 20 °C

Statistical stiffness values	DBM50	EME2	SMA
Mean	5975	6689	2115
Min	5682	6104	1975
Max	6536	7242	2166
Standard deviation	347	523	79
15th percentile	5719	6247	2072
Stiffness used in KENLAYER	5700	6200	2100

IV. DESIGN PRINCIPLES AND DETERIORATION ANALYSIS

In general, there are two predominant failure mechanisms frequently encountered on roadways: cracking of the asphalt or bound layers and rutting in the wheel tracks. Consequently, it is crucial to consider both failure modes in pavement design to ensure that the pavement can withstand these primary types of failure. However, this paper focuses solely on the fatigue criterion for pavement design and analysis. The key design parameter used is the tensile strains at the bottom of the surface, binder, and base layers. This is because fatigue cracking in bituminous layers is initiated by tensile strain rather than stress under repeated wheel loads.

A. Fatigue Equation

This study's pavement fatigue design is grounded in laboratory ITFT results. The fatigue equation incorporates the c_2 and f_2 factors derived solely from the laboratory fatigue tests, alongside an f_s shift factor is employed to make the transition from laboratory to field performance, as seen in (1) [26]. For asphalt pavements, shift factors reported in research range from 2 to 440. Analyzing the influence of individual factors has resulted in total shift factors between 10-20 or 440 [15, 16]. A shift factor of 77 was used, consistent with the ITFT data from the University of Nottingham [28, 29]. In another study, a shift factor of 42.4 was suggested to be more appropriate, highlighting the beneficial effects of polymer-modified binders on the fatigue life of PMA mixtures [30]. This aligns with the findings of another study proposing a shift factor of 45 for grouted macadam materials. To simplify the analysis, a shift factor of 45 has been selected.

$$N_f = f_s c_2 (\varepsilon_t)^{-f_2} \qquad (1)$$

where N_f is the allowable number of load repetitions to prevent fatigue cracking, ε_t is the tensile strain at the bottom of the asphalt layer, and f_s, c_2 and f_2 are constants. The regression coefficients c_2 and f_2 for the fatigue equation taken from the ITFT have been chosen for the damage analysis, as portrayed in Table II [27].

TABLE II. ITFT REGRESSION COEFFICIENTS FOR FATIGUE EQUATION

Mixture Type	Design Temperature	c_2	f_2	R^2
DBM50	20 °C	5.365×10^{14}	4.762	0.98
EME2	20 °C	8.532×10^{12}	3.906	0.96
SMA	20 °C	5.831×10^{14}	4.065	0.90

B. Deterioration Analysis Approach

For this deterioration analysis, an iterative process has been developed based on several previous studies [4, 31]. The initial step involves determining the strains at the bottom of each layer, ε_0, utilizing the initial stiffnesses (E_0) from the KENLAYER. These parameters are then used to calculate the life of each sub-layer (N_0) according to (1). The failure criterion is defined as a stiffness reduction of 95%, 67%, and 71% at 20 °C for the DBM50, EME2, and SMA materials, respectively, based on the BWTFT results as reported in prior studies [17]. This represents the expected service life of each sub-layer, assuming a constant strain throughout the pavement's lifetime. However, as previously noted, this assumption does not accurately reflect the conditions observed in real-world road infrastructure. In the second step, an iteration interval is chosen. For this analysis, a 20% damage to the study layer is selected. This percentage, which necessitates 5 iterations, is adopted to conserve time. Nevertheless, it is evident that augmenting the number of iterations produces more precise outcomes. In the initial iteration, the sub-layer with the lowest remaining life is identified, yielding $N_{f,1}$. The life interval is then computed by multiplying $N_{f,1}$ by 20% and denoted as I_1, representing the life consumed in this first iteration. The damage degree of each sub-layer (I_1/N_0) is subsequently calculated, and new stiffness (E_1) values for each sub-layer are determined using the damage degree and stiffness reduction data from the BWTFT after $0.2\times N_{f,1}$ load applications. By utilizing the updated E_1 values in KENLAYER, new strains at the bottom of the sub-layers (ε_1) are obtained. These strains are found to be higher than the previously calculated values, indicating that the pavement has sustained 20% damage.

To perform the subsequent iteration for the next 20% of the pavement's service life ($N_{f,2}$), the pavement's remaining lifespan needs to be recalculated using (1) with the previously obtained strain value ε_1 ($I_2 = N_{f,2} \times 0.2$). Similarly, the degree of damage and stiffness reduction should be computed to determine the new modulus E_2 and corresponding strains ε_2. The strains ε_2 are expected to be higher than the previously calculated ε_1 as the pavement has already experienced 40% damage. This process is repeated five times until the pavement's full functional lifespan is exhausted. The pavement's total service life (N_f) is quantified as the cumulative sum of all lifespan intervals (ΣI_i).

V. RESULTS AND DISCUSSION

A. Evolution of Strain in a Pavement

Figures 7 and 8 illustrate the strain evolution in pavements 1 and 2, where the binder and base course materials are DBM50 and EME2, respectively. Similarly, Figure 9 depicts the strain development in pavement 3, which features an SMA surface course. These Figures display the predicted strain variations throughout the pavement depths, corresponding to different degrees of deterioration from 100% to failure. It is evident that the strain undergoes changes over the pavement's service life for all three material configurations. The largest strain variation is observed in pavement structure 1 with the DBM50 base, followed by pavement structure 2 with the EME2 base, whereas pavement structure 3 with the SMA surface exhibits the smallest strain changes.

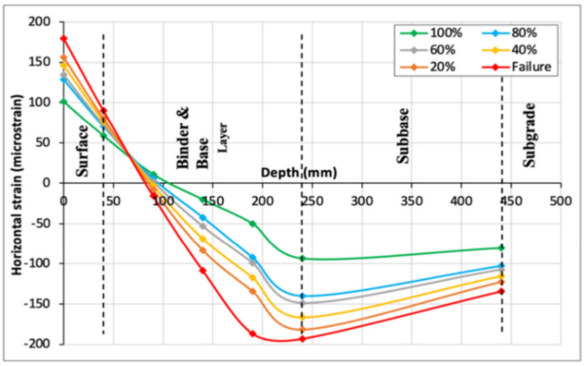

Fig. 7. Strain evolution with the DBM50 binder and base.

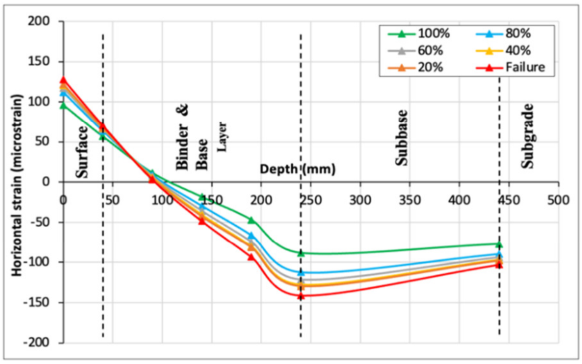

Fig. 8. Strain evolution with the EME2 binder and base.

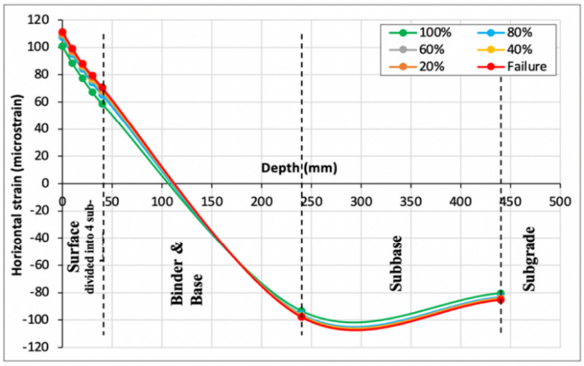

Fig. 9. Strain evolution with the SMA surface.

B. Life Ratio of Layers Predicted for Traditional and Deterioration Analysis

The analysis presented in Figure 10 demonstrates that the EME2/DBM50 life ratio varies considerably depending on the approach utilized. A traditional analysis yielded a ratio of 0.96, whereas a deterioration analysis resulted in a higher ratio of 1.51. This suggests that the deterioration modeling approach reveals a more significant advantage for PMA compared to the traditional method. However, the analysis technique is limited in its application to the SMA surface course in pavement structure 3, as it is focused solely on bottom-up cracking. Additionally, a direct material comparison cannot be made in this case.

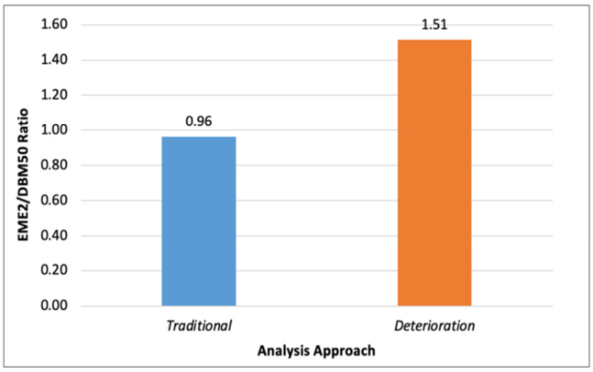

Fig. 10. Life ratio of EME2 to DBM50 predicted.

VI. CONCLUSIONS

This study conducted a deterioration analysis of pavements incorporating Polymer-Modified Asphalt (PMA) materials, including 20 mm Dense Bitumen Macadam (DBM50), 14 mm Enrobé à Module Élevé 2 (EME2), and 10 mm Stone Mastic Asphalt (SMA). The analysis utilized the KENLAYER program, with three different pavement structure scenarios modeled. An iterative approach was developed, leveraging average stiffness reduction data from the Beam Wheel Tracker Fatigue Test (BWTFT), stiffness modulus values from the Indirect Tensile Stiffness Modulus (ITSM) test, and fatigue regression coefficients from the Indirect Tensile Fatigue Test (ITFT).

The deterioration analysis facilitated more realistic modeling of pavements, demonstrating that strain variations occur throughout the pavement's lifespan. Notably, the iterative process results indicated that pavement structure 1 with a DBM50 base exhibited the highest strain variation, followed by pavement structure 2 with an EME2 base, and pavement structure 3 with an SMA surface showing the least strain variation.

Regarding pavement lifespan, the iterative deterioration analysis enabled a more accurate comparison of different materials under realistic conditions. It was found that, under deterioration analysis, the advantages of using PMA become substantially more pronounced than with traditional approaches. Finally, based on the life ratio of EME2 to the conventional DBM50 base material, the ratio calculated through traditional analysis was approximately 1.0, while with deterioration analysis, this ratio increased to 1.51.

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