Evaluating the Performance of High Modulus Asphalt Concrete Mixture for Base Course in Iraq

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ABSTRACT

In the 1980s, the French Administration Roads LCPC developed high modulus mixtures (EME) by using hard binder. This type of mixture presented good resistance to moisture damage and improved mechanical properties for asphalt mixtures including high modulus, good fatigue behaviour and excellent resistance to rutting. In Iraq, this type of mixture has not been used yet. The main objective of this research is to evaluate the performance of high modulus mixtures and comparing them with the conventional mixture, to achieve this objective, asphalt concrete mixes were prepared and then tested to evaluate their engineering properties which include moisture damage, resilient modulus, permanent deformation and fatigue characteristics. These properties have been evaluated using indirect tensile strength, uniaxial repeated loading and repeated flexural beam tests. EME mixes were found to have improved fatigue and permanent deformation characteristics, also showed more resistance to moisture damage than conventional mix by 9.3 percent and the resilient modulus at temperature 60 ^{o}C increased by 63 percent. The general theme viewed from the results of this study has added to local knowledge the ability to produce more durable asphalt concrete mixtures with better serviceability using EME mixes.

Key words: high modulus asphalt mixture, fatigue, rutting.

تقييم اداء الخلطة الاسفلتية العالية الجساءة لطبقة الاساس في العراق

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الخلاصة

في سنة ١٩٨٠، طورت المؤسسة الفرنسية للطرق (LCPC) الخلطات العالية الجساءة، (EME) بواسطة استخدام اسفلت بتدرج صلب. هذا النوع من الخلطات يعطي مقاومة جيدة لضرر الرطوبة و يحسن الخواص الميكانيكية للخلطات الاسفلتية التي تشمل جساءة عالية،تصرف جيد للكلل ومقاومة ممتازة للتخدد. هذا النوع من الخلطات لم يستخدم بعد في العراق. الهدف الرئيسي من هذا البحث هو تقييم اداء الخلطات العالية الجساءة ومقارنتها بالخلطات الاعتيادية، للوصول لهذا الهدف، تم تحضير خلطات الخرسانة الاسفلتية وفحصها لتقييم الخصائص الهندسية. هذه الخصائص تم تقييمها بأستخدام فحوصات نسبة الشد غير المباشر، الحمل المتكرر احادي المحور وفحص الانحناء المتكرر. أنشأت خلطات ال EME لتحسين خصائص الكلل و التشوهات الدائمية، وكذلك اظهرت مقاومة للضرر الحاصل بسبب الرطوبة اكثر من الخلطات الاعتيادية بنسبة 9,3 % و ومعامل المرونة عند درجة حرارة 60 ^{o}س ازداد بنسبة 63 %. اضيف الى المعرفة المحلية الموضوع العام الذي يعرض من نتائج هذه الدراسة هو القابلية لانتاج خلطات خرسانية اسفلتية بتحمل اعلى مع خدمة افضل بأستخدام خلطات EME.

الكلمات الرئيسية: الخلطة الاسفلتية العالية الجساءة، الكلل، التخدد.

1. INTRODUCTION

Several distresses hamper the performance of flexible pavements in Iraq and result in premature failure. In flexible pavements, the primary forms of distress are fatigue cracking and rutting. These distresses manifest themselves most of the time due to construction material quality, poor maintenance, and improper structural design. The necessity to increase the service life of pavement subjected to high traffic volume imposes the use of asphalt mixture with high performance. This led to the development of EME (Enrobés à Module Elevé) or High- Modulus Asphalt (HiMA) in the 1980s by French road administration (LCPC), bitumen producers and road contractors. In view of this, the need for the use of EME mixture to improve mechanical properties including high resilient modulus, good moisture damage resistance, good fatigue behaviour and excellent resistance to rutting.

With this purpose in mind, the primary objective of this study is to evaluate the performance of high modulus mixtures and compare them with the conventional mixture for base course based on the following tests, Marshall properties (mix design parameters), indirect tensile test (moisture damage resistance), uniaxial repeated load test (resilient modulus and permanent deformation) and repeated flexural beam test (fatigue characteristics).

2. BACKGROUND

The specification for EME (High Modulus Mixture) is shown in specification NF P 98-140, **AFNOR, 1999**. There are two classes of EME mixture, EME Class 1 (EME1) and Class 2 (EME2). Both mixtures are designed to have high modulus, but EME2 has a higher asphalt content requirement and specified for most heavily trafficked roads. As noted by **Dariuze, et al., 2010**, bitumen with harder grades are used, mainly 10/20, 15/25, and even 5/15; polymer modified bitumen (PMB) is also used. In **Sanders, and Nunn, 2005**, the UK Highways Agency recommends a 10/20 and 15/25 grade bitumen as binder for use in EME base/binder coarse asphalt mixtures while **Denneman, et al., 2011** and **Vaitkus, et al., 2013** used hard binder (20-30).

EME is designed to satisfy criteria determined using laboratory tests developed by French Administration Roads LCPC for **Delorme, et al., 2007** to measure the performance properties of laboratory compacted test specimens in respect of compatibility, water sensitivity, deformation resistance, stiffness and fatigue cracking resistance. The binder content is controlled by a richness modulus (K), which is a function of the mass of the soluble binder expressed as a percentage of total dry mass of aggregate, the specific surface area of aggregate and the density of the aggregate, **AFNOR, 1999**. **Lee, and Park, 2007** found out that the rutting resistance of HMAC is twice higher than that of hot mix asphalt, and the fatigue resistance is 5–10 times higher by using polymer modified 20/30 bitumen. **Rohde, et al., 2008** analyzed the HMAC to find that the accumulated permanent deformation 6.7% after 10,000 loading cycles. On the other hand, the comparison mixture with conventional binder (50/70) presented permanent deformation higher than 23% just after 1,300 cycles. According to **Hussain Bahia, et al., 2013**, the HMABs can be effective in reducing layer thickness. One grade increase in the continuous high temperature performance grade reduces the thickness of asphalt layer by 0.065 inch without an increase in total permanent deformation. The asphalt layer thickness is reduced by 0.89 to 4.4 inches by replacing the neat binder with HMABs produced in this study. The results of **Grobler, et al., 2011** indicated that the higher stiffness and increase in fatigue life (2 to 3 times more than normal asphalt) a decrease in asphalt thickness up to 30% is possible. In **Hussain Bahia, et al., 2013**, there are still some barriers which prevent hard binder from being used as a worldwide road construction material. First of all, the viscoelastic properties of hard binder are not well understood. Secondly, because hard binder is stiff with a higher viscosity, workability and thermal cracking resistance become critical factors that need further evaluation. Finally, the effectiveness of hard binder as wearing course and binder course materials is still debated and needs further evaluation (**Hussain Bahia, et al., 2013**).

3. MATERIAL CHARACTERIZATION

Asphalt cement, aggregate, and filler used in this work have been characterized using routine type of tests and the results were compared with State Corporation for Roads and Bridges specifications (SCRB, 2003). All experimental works have been performed in highway Materials and Construction Materials Laboratories in Civil Engineering Department, College of Engineering, University of Baghdad.

3.1 Asphalt Cement

In this study, two asphalt grades are considered: grade (20-30) and grade (40-50). Grade (20-30) type AIWIN was brought from Jordan and grade (40-50) was brought from Dora refinery, south-west of Baghdad. The physical properties of the asphalt cement are shown in Table (1).

3.2 Aggregate

EME is typically produced using fully crushed fractured aggregate for base course but rounded aggregate used in Iraqi practice. The aggregate used in this work was obtained from Amanat Baghdad asphalt concrete mix plant located in Taji, north of Baghdad, its source is Al-Nibaie quarry. This type of aggregate is widely used in Baghdad city for asphaltic mixes. EME shall be designated as EME 0/10, EME 0/14 or EME 0/20 (minimal/maximal particle size in mm, where 0 represents the 0.075 mm) according to aggregate size as specified in **Sanders, and Nunn, 2005**. In this study, the appropriate grading for the base course based on **SCRB, 2003** is EME 0/20. The coarse and fine aggregates used in this work were sieved and recombined in the proper proportions to meet the base course gradation as required by SCRB specification **SCRB, 2003**. The aggregate gradation properties are presented in Table (2) and gradation curve is shown in **Fig.1**. Routine tests were performed on the aggregate to evaluate their physical properties. The results together with the specification limits as set by the **SCRB, 2003** are summarized in Table (3). Tests results show that the chosen aggregate met the **SCRB, 2003** specifications.

3.3 Filler

The filler is non-plastic materials that passes sieve No.200 (0.075mm). Mineral filler used in this work is limestone dust obtained from Amanat Baghdad asphalt concrete mix plant; its source is the lime factory in Karbala governorate, south east of Baghdad. The physical properties of the used filler are presented in **Table 4.**

4. EXPERIMENTAL WORK

The experimental work was started by Marshall Test to find optimum asphalt content for conventional and high modulus asphalt mixtures (EME). Indirect tensile test to evaluate the moisture damage resistance and the mechanical properties which include resilient modulus, permanent deformation and fatigue characteristics. The mechanical properties have been evaluated using uniaxial repeated loading and repeated flexural beam tests.

4.1 Marshall Test

To obtain the optimum asphalt content (O.A.C) for base course, Marshall specimens were prepared according to the Marshall method as outlined in Asphalt Institute manual (series No.2 ,1981) using 75 blows (SCRB, 2003) of the automatic Marshall compactor on each side of specimen. The specimens were evaluated for Marshall Stability, flow value, percent air voids (AV) and percent voids in mineral aggregate (VMA).The optimum asphalt content for HMAC and conventional mixtures were 4.6% and 3.8% respectively. Table (5) presents the mixtures properties at optimum asphalt contents.

4.2 Indirect Tensile Test

The moisture susceptibility of the bituminous concrete mixtures was evaluated according to (ASTM- D-4867-96). The result of this test is the indirect tensile strength (ITS) and tensile strength ratio (TSR). In this test, a set of specimens was prepared for each mix according to Marshall Procedure and compacted to 7±1 % air voids using different numbers of blows per face, varying from 17 to 19 (targeted air voids content were prepared to voids is not meant to mimic the actual field conditioning process but to accelerate the moisture damage in a manner that can be measured under laboratory conditions). The set consists of six specimens and was divided into two subsets, one set (control) was tested at 25°C and the other set (soaked) was subjected to one cycle of freezing and thawing then tested at 25°C. The test is shown below in **Fig. 2**. It involved loading the specimens with compressive load at a rate of 50.8 mm/min acting parallel to and along the vertical diametric plane through 0.5 in. wide steel strips which are curved at the interface with the specimens. These specimens failed by splitting along the vertical diameter. The indirect tensile strength calculated according to Eq. (1) of the soaked specimens (ITS_c) is divided by that of the control specimens (ITS_d), which gives the tensile strength ratio (TSR) as the following Eq. (2).

$$ITS = \frac{2P}{\pi t D} \quad (1)$$

$$TSR = \frac{TSR\ c}{TSR\ d} \quad (2)$$

Where

ITS= indirect tensile strength
P = ultimate applied load
t = thickness of specimen
D = diameter of specimen
Other parameters are defined previously

4.3 Uniaxial Repeated Loading Test

The uniaxial repeated loading tests were conducted for cylindrical specimens, 101.6 mm (4 inch) in diameter and 203.2 mm (8 inch) in height, using the pneumatic repeated load system (shown in **Fig. 3**). In these tests, repetitive compressive loading with a stress level of 20 psi was applied in the form of rectangular wave with a constant loading frequency of 1 Hz (0.1 sec. load duration and 0.9 sec.

rest period) and the axial permanent deformation was measured under the different loading repetitions. All the uniaxial repeated loading tests were conducted at 20, 40 and 60 °C. The specimen preparation method for this test can be found elsewhere **Albayati, 2006**. The permanent strain (ε_p) is calculated by applying the following equation:

$$\varepsilon_p = \frac{Pd \times 10^6}{h} \tag{3}$$

Where:

ε_p = axial permanent microstrain

pd = axial permanent deformation

h = specimen height

Also, throughout this test the resilient deflection is measured at the load repetition of 50 to 100, and the resilient strain (εr) and resilient modulus (Mr) are calculated as follows:

$$\varepsilon_r = \frac{r_d \times 10^6}{h} \tag{4}$$

$$M_r = \frac{\sigma}{\varepsilon r} \tag{5}$$

Where:

ε_r = axial resilient microstrain

rd = axial resilient deflection

h = specimen height

M_r = Resilient modulus

σ = repeated axial stress

εr = axial resilient strain

The permanent deformation test results for this study are represented by the linear log-log relationship between the number of load repetitions and the permanent microstrain with the form shown in Eq. (6) below which is originally suggested by **Monismith, et al., 1975** and **Barksdale, 1972.**

$$\varepsilon_p = a\, N^b \tag{6}$$

Where:

ε_p = permanent strain

N = number of stress applications

a = intercept coefficient

b = slope coefficient

Once regression coefficients had calculated (a and b), the permanent deformation coefficients Alpha (α) and Mu (μ) were computed using the relationship given in Eq. (7) and (8) (Huang, 1993):

$$\alpha = 1\text{-}b \qquad (7)$$

$$\mu = \frac{a\ b}{\varepsilon r} \qquad (8)$$

Where (μ) is the permanent deformation parameter representing the constant of proportionally between permanent strain and resilient strain (i.e. plastic strain at N =1) and α is a permanent deformation parameter indicating the rate of decrease in incremental permanent deformation as the number of load applications increases.

4.4 Flexural Beam Fatigue Test

Within this study, third-point flexural fatigue bending test was adopted to evaluate the fatigue performance of asphalt concrete mixtures using the pneumatic repeated load system, this test was performed in stress controlled mode with flexural stress level varying from 15 to 45 psi applied at frequency of 2 Hz with 0.1 sec loading and 0.4 sec unloading times and in rectangular waveform shape. All tests were conducted at 10°C on beam specimens 76 mm (3 in) x 76 mm (3 in) x 381 mm (15 in) prepared according to the method described in **Alkhashab, 2009**. In the fatigue test, the initial tensile strain of each test has been determined at the 50th repetition by using Eq. (9) shown below and the initial strain was plotted versus the number of repetition to failure on log scales, collapse of the beam was defined as failure, the plot can be approximated by a straight line and has the form shown below in Eq. (10).

$$\varepsilon_t = \frac{\sigma}{E_S} = \frac{12d\Delta}{3L^2-4a^2} \qquad (9)$$

$$Nf = K_1\, \varepsilon t^{-K2} \qquad (10)$$

Where:

ε_t = Initial tensile strain
σ = Extreme flexural stress
Es = Stiffness modulus based on centre deflection.
h = Height of the beam
Δ = Dynamic deflection at the centre of the beam.
L = Length of span between supports.
a = Distance from support to the load point (L/3)
N_f = Number of repetitions to failure
k_1 = fatigue constant, value of N_f when = 1
k_2 = inverse slope of the straight line in the logarithmic relationship

5. TEST RESULTS AND DISCUSSION

5.1 Asphalt Concrete Mixture Properties

The optimum asphalt content (O.A.C) and Marshall stability for HMAC were more than conventional mixture by 21% and 31% respectively. Therefore, the highest Marshall stability, flow and voids in mineral aggregate are achieved with the stiff binder type (20-30).

5.2 Durability Performance

The durability of HMAC is assessed in France using an unconfined compressive test (EN 12697-12) on moisture conditioned specimens (Duriez test). In this study, the modified Lottman test in accordance with ASTM D4867 is generally used for this purpose.

The Tensile Strength Ratio (T.S.R.) test results are shown in Table (6) for conventional and HMAC mixture at optimum asphalt content (O.A.C). Generally a minimum TSR of 0.70 is recommended. The TSR results corresponding to HMAC and “conventional” specimens were 0.79 and 0.72, respectively, showing that both mixtures present good resistance to moisture damage. The results showed that the high modulus asphalt mixture had a resistance to moisture damage more than the conventional asphalt mixture approximately by 9.3 %.

5.3 Resilient Modulus

Table (7) and **Fig. 4** show the values of Mr for the conventional and HMAC mixtures. When using HMAC instead of conventional mixture at (O.A.C), the value of (Mr) increased by 29, 102 and 63% for temperature 20, 40 and 60 ^{0}C respectively. In case of decreasing the asphalt content for HMAC mixture from 4.6 to 4%, the value of (Mr) increased by 13, 17 and 11% for temperature 20, 40 and 60 ^{0}C respectively.

5.4 Permanent Deformation

Based on the data shown in Table (8), **Fig. 5** and **Fig. 6**, it appears that the hard binder contents have influence on the plastic parameter of the material as characterized by the (α) and (μ) values.When using HMAC instead of conventional mixture, the value of (μ) decreased and the variable (α) has an opposite effect.

5.5 Flexural Fatigue

For the hard binder used in this study, increase asphalt content significantly affect the number of cycles to failure NF and provide an increased level of protection against cracking due to repetitive loading because higher asphalt content increases the thickness of the binder film between aggregates, which results in lower stress in the binder film. Fatigue cracking coefficient (K_1) and exponent (K_2) are presented in Table (9) for the conventional mix and mixes for HMAC. Values of k_1 and k_2 can be used as indicators of the effects of stiff binder on the fatigue characteristics of a paving mixture. The flatter the slope of the fatigue curve, the larger the value of k_2 which indicates a potential for longer fatigue life. On the other hand, a lower k_1 value represents a shorter fatigue life. As can be seen from **Fig. 7** as the stiff binder used with high asphalt content the k_2 value increases and the k_1 value decreases. These results highlight the improvement in fatigue resistance for mixes with high content of stiff binder. The fatigue life increases for HMAC more than conventional mixture as illustrated in **Fig. 8**.

6. MINIMUM BINDER CONTENT using RICHNESS FACTOR (K)

The French specification includes two classes of HMAC (EME) mixes for base course: Class 1 for light traffic, and Class 2 for heavy traffic. To find out the class of the HMAC mixture used in this study, the equation of the minimum binder content is applied as follows:

$$TL = K * \alpha * \sqrt[5]{\sum} \qquad (11)$$

The correction coefficient (α) was computed by finding the effective specific gravity of aggregate (Gse) and ($\sum$) value depending on the aggregate grading. The minimum binder content (TL) is the percentage by the mass of aggregate. Therefore, to find the percentage of binder content by the mass of the total mix (P_b), the following equation is used, **AFNOR, 2002, Denneman et al., 2011** and **Sanders, and Nunn, 2005**:

$$TL = \frac{100*Pb}{100-Pb} \qquad (12)$$

The result obtained from the above equations is illustrated in Table (10). The optimum asphalt content for HMAC mixture is (4.6 percent). Therefore, the content of O.A.C-0.6 (4 percent) is less than the minimum binder content of Class 2 (5.3 percent) so the mixture of HMAC in this study can be classified as Class 1.

7. EVALUATION of PRESENT SERVICEABILITY INDEX (PSI)

In order to reduce the risk of unsatisfactory pavement performance to an unacceptable level, engineers must be able to reliably predict pavement behavior with time. In this study, VESYS 5W software is used to predict the present serviceability index with time. An analysis period of 20 years is used in the analysis. For the practical example, the geometry of the pavement structure is shown in **Fig. 9**. The Mean Air Pavement Temperature (MAPT) is required as an input value. The assumed MAPT was 40^0C, which temperature expected to occur in the middle of the asphalt concrete pavement layer during the hot summer seasons in Iraq.

VESYS 5W needs to be provided with parameters to run the analyses in form of inputs data. VESYS 5W software has been successfully used to analyze the asphalt pavement performance under field traffic and under accelerated pavement testing loads. VESYS 5W needs to be provided with parameters to run the analyses in form of inputs data. These input parameters include material properties of the layers, thickness, traffic data and environmental conditions (Appendix **A**).

The VESYS analysis procedure uses advanced mechanistic concepts to predict the behavior and performance of flexible pavements. Strain and deflection responses are computed, and then used in conjunction with failure criteria to predict pavement distress in terms of cracking, rutting and roughness. Distress is used to define pavement performance in terms of life history of the present serviceability index (PSI). All of the components of the design procedure have been formulated to take into account the inherent variability in traffic estimates, materials properties, and environmental conditions and in the many forms of construction practices used (VESYS 5W user Manual 2003).

These input parameters include (K_1, K2, α, μ and Mr) which can be obtained from fatigue and rutting test. The Percent Serviceability Index (PSI) relationship with time is shown in **Fig. 10**. For a pavement section with 20 years design life, the drop in serviceability index value for the pavement section with EME mixes was less than 1 whereas for conventional mixes the corresponding value is 2.5.

8. CONCLUSIONS

Within the limitations of materials and testing program used in this work, the following principal conclusions are made based on the findings of the investigations:

1. Using HMAC instead of conventional mixture at (O.A.C) lead to increased the resilient modulus (Mr) by 29, 102 and 63% for temperature 20, 40 and 60 ^{0}C respectively. Further decrease in asphalt content beyond the optimum resulted in increased of Mr value.
2. The EME has an excellent resistance to rutting as compared with the conventional mixture at their optimum asphalt content, which was 4.6 for the former and 3.8% for the later. The (α) value for EME mix was more than that of conventional mix by 4.1, 39.7 and 56.7 % for temperature 20, 40 and 60 ^{0}C respectively.
3. Fatigue test results reflected better performance for the EME mix against fatigue cracking as compared to the conventional mix, the fatigue parameter (K1) for the conventional mixture was (6.906 E-10) whereas for EME the corresponding value was (6.777 E-10) for O.A.C (4.6 %). The K2 (inverse slope of fatigue line) for the EME was 3.731 at 4.6 % asphalt content whereas for the conventional mix the value was 3.704. The EME mix with 4% asphalt content introduced less resistance to fatigue cracking as compared with conventional mixture.
4. The results of indirect tensile test showed that the HMAC mix had more resistance to moisture damage than conventional mix approximately by 9.3 % so the HMAC passed the requirement for EME specification for durability performance.
5. The optimum asphalt content (O.A.C) and Marshall stability for EME was more than that of conventional mixture by 21% and 31% respectively. Therefore, the highest Marshall stability, flow and voids in mineral aggregate are achieved with the use of stiff bitumen.
6. Based on minimum Richness factor (K) equation, the HMAC mixture for this study is EME Class1 with minimum binder content of 4 percent.
7. The drop in serviceability index value for the pavement section with EME mixes was less than 1 whereas for conventional mixes the corresponding value is 2.5. With this result in view, the use of EME mixes has added to local knowledge the ability to produce more durable asphalt concrete mixtures with better serviceability.

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10. NOMENCLATURE

a	intercept coefficient, dimensionless
a	distance from support to the load point (L/3), mm
Av	air voids, %
b	slope coefficient, dimensionless
D	diameter of specimen, in
Es	stiffness modulus based on centre deflection, N/mm^2
Gse	effective specific gravity of aggregate, gm/cm^3
h	height of the beam, mm
ITS	indirect tensile strength, psi
K	richness modulus, dimensionless
k_1	fatigue constant, value of N_f when = 1, dimensionless
k_2	inverse slope of the straight line in the logarithmic relationship, dimensionless
L	length of span between supports, mm
M_r	resilient modulus, psi
N	number of stress applications
N_f	number of repetitions to failure
P	ultimate applied load, Ib
P_b	percentage of binder content by the mass of the total mix, %
pd	axial permanent deformation
rd	axial resilient deflection
T	thickness of specimen, in
TL	percentage of binder content by the mass of aggregate, %
TSR	tensile strength ratio, %

VMA voids in mineral aggregate, %
Δ dynamic deflection at the centre of the beam
α correction coefficient, dimensionless
ε_p axial permanent strain, microstrain
ε_r axial resilient strain, microstrain
ε_t initial tensile strain, microstrain
σ repeated axial stress, psi

Table 1. Properties of asphalt cement.

Properties before TFOT					
Test	ASTM Designation	Penetration grade 40-50		Penetration grade 20-30	
		Results	SCRB Specification (2003)	Results	EN-12591 Specification
Penetration at 25 ^{0}C, 100gm, 5 sec (0.1mm)	D5	44	40-50	24	20-30
Ductility at 25 ^{0}C, cm	D113	116	>100	18	-----
Flash Point, ^{0}C	D92	260	232 Min.	300	240 Min.
Softening Point, ^{0}C	D36	48	-----	61.5	55-63
Penetration Index	-----	-1.98	-----	-0.27	+0.7 Max. -1.5 Min.
Properties after TFOT					
Penetration at 25 ^{0}C	D5	29	29	18	-----
Retained Pen. %	------	66	55 (Min.)	75	55(Min.)
Change of Mass, %	------	-0.057	0.5 (Max.)	-0.074	0.5(Max.)
Ductility at 25 ^{0}C, cm	D113	43	25	13	-----
Softening Point, ^{0}C	D36	50.6	-----	68.4	-----
Increase in Softening Point, ^{0}C	------	2.6	-----	6.9	8 (Max.)

Table 2. Percent pass by weight of selected aggregate gradation

(25 mm nominal maximum size, base course).

EME 0/20			SCRB (R/9, 2003)			
Sieve size (mm)	% passing by weight		English Sieve.	Standard Sieves (mm)	% passing by weight	
	Selected Limit	Specification range			Selected Limit	Specification range
31.5	100	100	1½"	37.5	100	100
20	92	90-99	1"	25	95	90-100
14	78	75-95	3/4"	19	89	76-90
10	66	60-90	1/2"	12.5	72	56-80
6.3	59	42-75	3/8"	9.5	65	48-74
4	-	-	No.4	4.75	47	29-59
2	28	20-35	No.8	2.36	30	19-49
0.25	13	8-18	No.50	0.3	14	5-17
0.063	7	5-9	No.200	0.075	7	2-8

Table 3. Physical properties of aggregates.

No.	Laboratory Test	ASTM Designation	Test Results	SCRB Specification (2003)
Coarse Aggregate				
1	Apparent Specific Gravity	C-127	2.678	-
2	Bulk Specific Gravity	C-127	2.61	-
3	Water Absorption,%	C-127	0.21	-
4	Fractured pieces,%	D-5821	96	90 Min
5	Flat & Elongated particles,%	D-4791	5	10 Max
6	Percent Wear (Los Angeles Abrasion),%	C-131	17.5	30 Max
7	Soundness Loss by Magnesium Sulfate solution, %	C-88	3.83	18 Max
8	Clay Lumps & Friable Particles,%	C-142	0.6	3 Max
Fine Aggregate				
1	Apparent Specific Gravity	C-128	2.683	-
2	Bulk Specific Gravity	C-128	2.621	-
3	Water Absorption,%	C-128	0.4	-
4	Fine Aggregate Angularity,%	AASHTO TP33	54.78	-

5	Sand Equivalent,%	D-2419	68.45	45 Min.
6	Clay Lumps & Friable Particles,%	C-142	2.85	3 Max

Table 4. Physical properties of fillers.

Property	Test Result
Specific gravity	2.72
%Passing Sieve No.200 (0.075 mm)	96

Table 5. Marshall properties.

Marshall property	Mixture Type		Specification requirements (SCRB, 2003)
	Conventional	HMAC	
Binder content, %	3.8	4.6	3-5.5
Stability ,kN	13.40	17.50	5 Min.
Flow, mm	2.50	3.30	2-4
Air Voids, %	3.50	3.20	3-6
VMA, %	13.05	14.20	12 Min.

Table 6. Indirect tensile strength results.

Grade Type	I.T.S. Unconditioned specimens (psi)	I.T.S. Conditioned specimens (psi)	TSR (%)
20-30	154.09	121.84	79.07
40-50	146.51	104.46	71.70

Table 7. Resilient modulus test results.

Temperature ^{0}C	Mixture Type		
	Conventional (3.8%)	HMAC (4%)	HMAC (4.6%)
20	194048	283712	250853
40	114518	269924	231264
60	69050	125155	112640

Table 8. Plastic parameter results (Rutting test).

Mixture Type		Conventional 3.8%			HMAC 4%			HMAC 4.6%		
Temp. ^{o}C		20	40	60	20	40	60	20	40	60
Variables	α	0.752	0.539	0.386	0.810	0.784	0.669	0.783	0.753	0.605
	μ	0.330	0.590	0.799	0.310	0.398	0.551	0.301	0.377	0.537

Table 9. Fatigue tests result.

Fatigue Coefficient	Mixture Type		
	Conventional (3.8%)	HMAC (4%)	HMAC (4.6%)
K_1	6.906E-10	1.467E-7	6.777E-10
K_2	3.704	2.874	3.731

Table .10 Minimum binder content based on EME class.

EME Class	K min.	α	$\sum$	TL min. percent	P_b min. percent
Class 1	2.5	1.0192	11.001	4.12	4
Class 2	3.4	1.0192	11.001	5.6	5.3

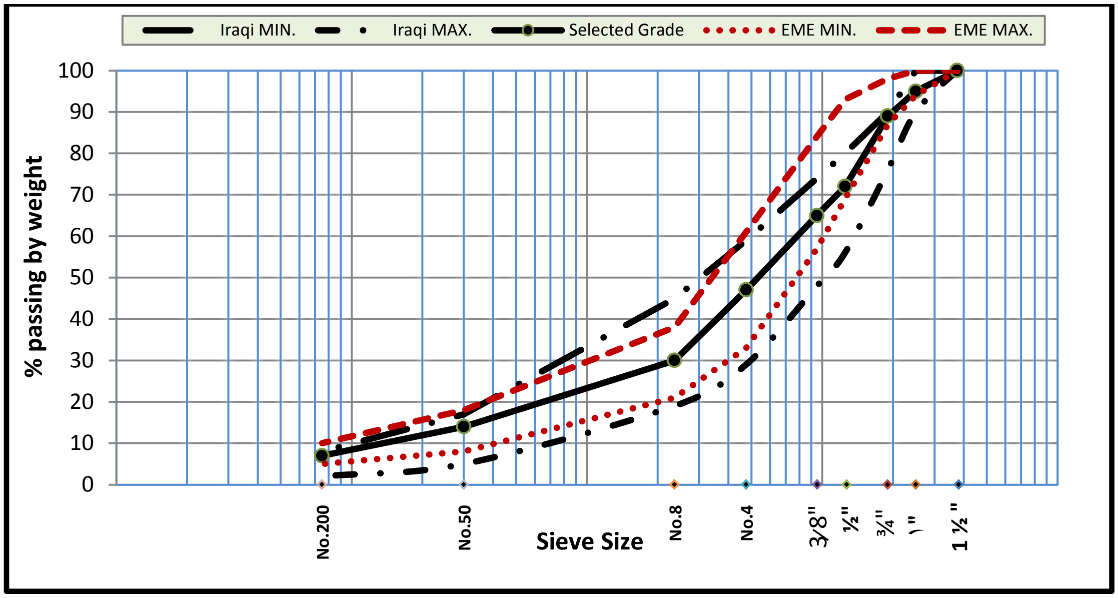

Figure 1. Aggregate gradation.

Figure 2. Photograph for ITS test.

Figure 3. Photograph for the PRLS.

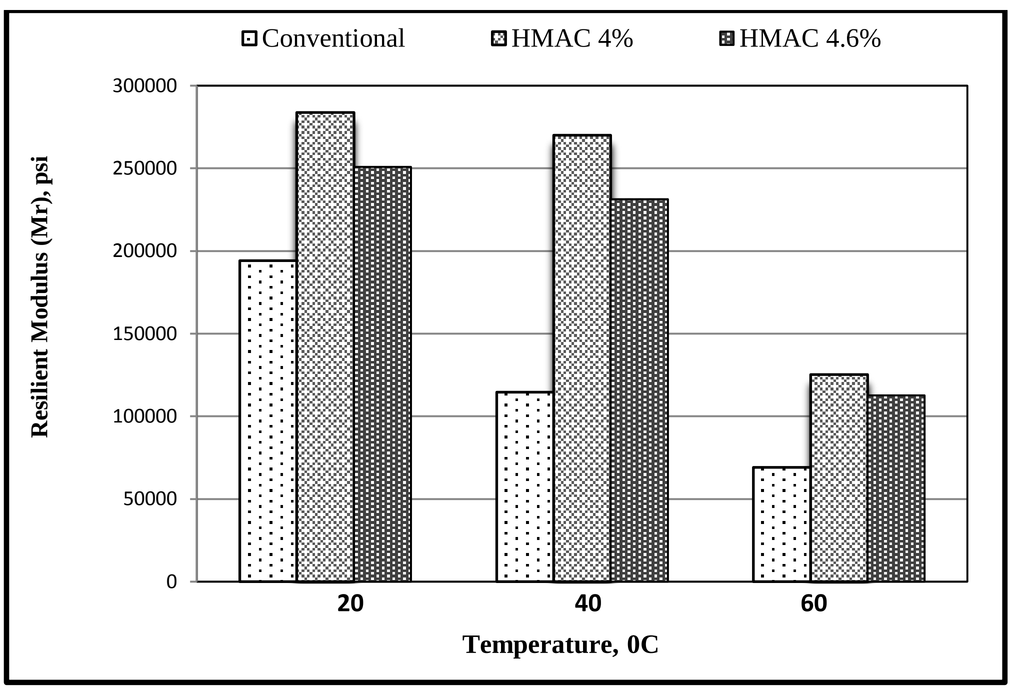

Figure 4. Effect of stiff binder on resilient modulus (Mr).

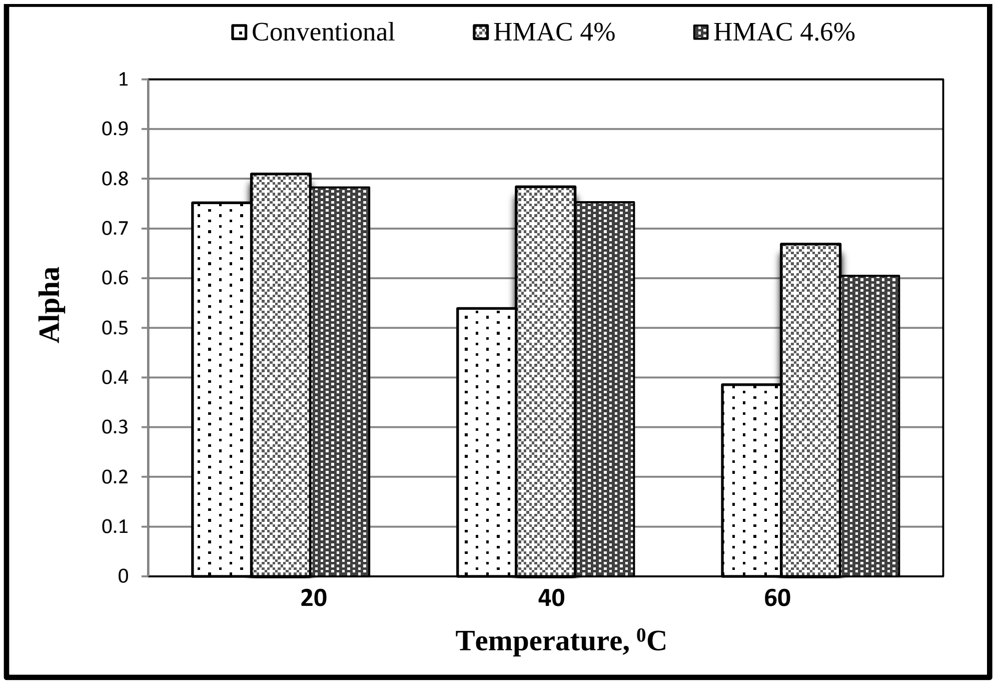

Figure 5. Effect of stiff binder on plastic parameter alpha (α).

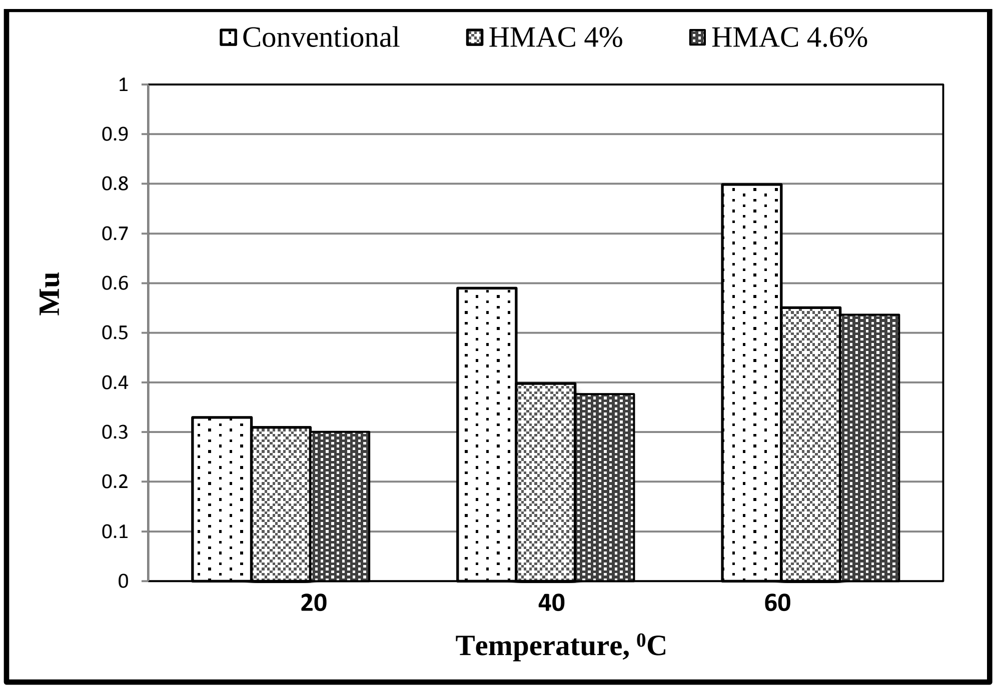

Figure 6. Effect of stiff binder on plastic parameter Mu (μ).

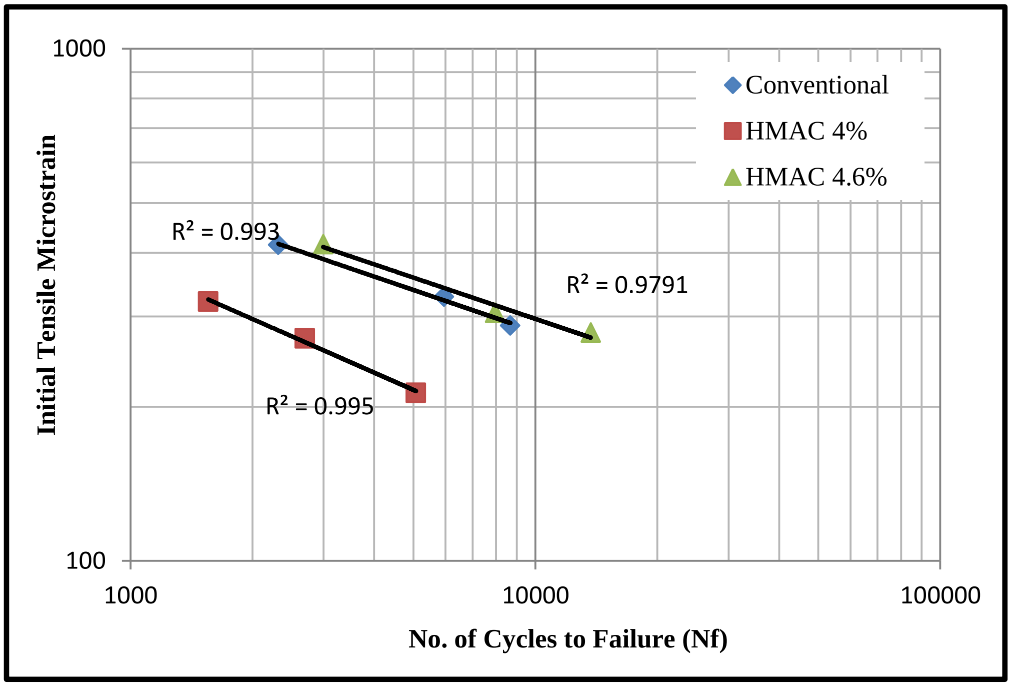

Figure 7. Effect of stiff binder content on fatigue performance.

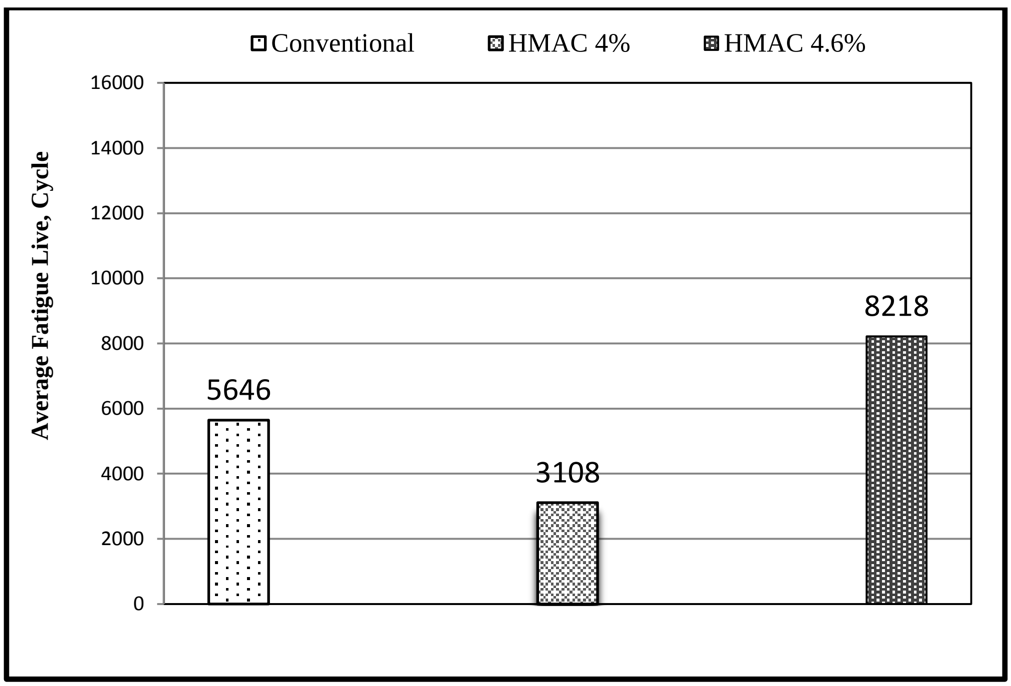

Figure 8. Effect of stiff binder content on fatigue life.

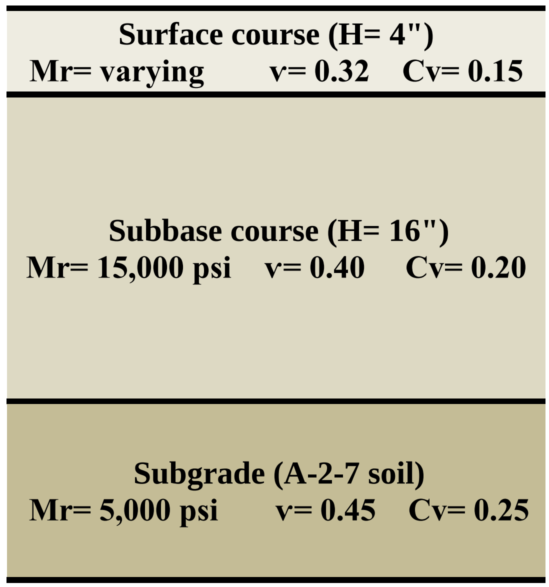

Figure 9. Geometry of the pavement structure.

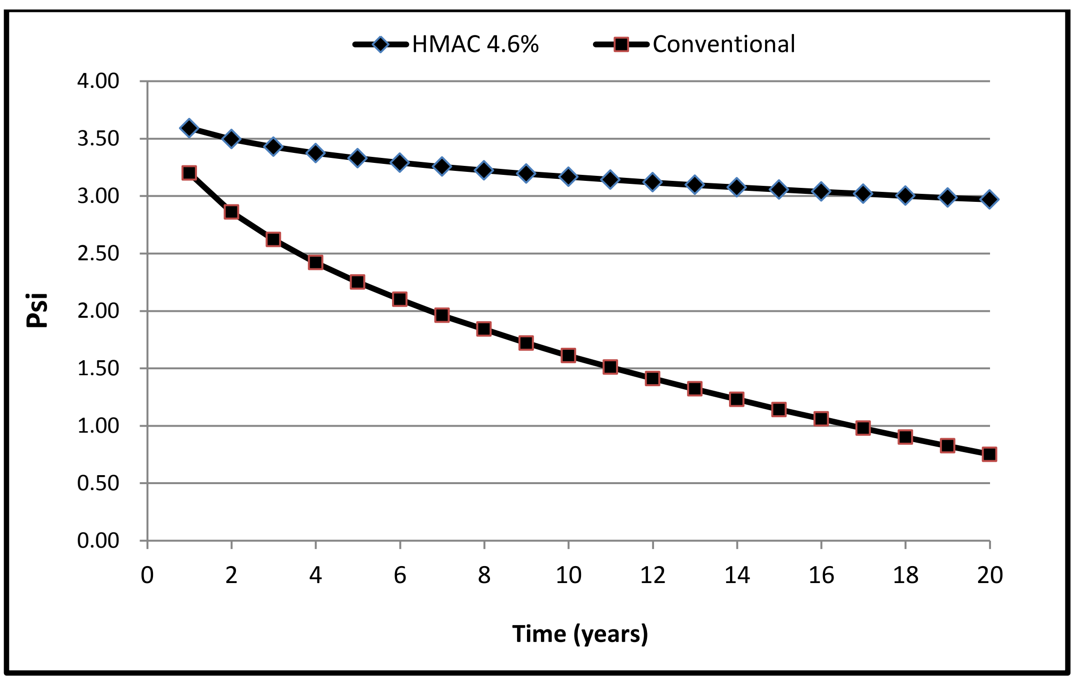

Figure 10. Percent serviceability index (PSI).