

Investigation of Mechanical Properties of Short- and Long-term Aged Asphaltite Modified Asphalt Mixtures

Mehmet YILMAZ*, Baha Vural KÖK, Necati KULOĞLU

Firat University, Faculty of Engineering, Department of Civil Engineering, 23119, Elazığ, Turkey

*mehmetyilmaz@firat.edu.tr

Abstract

In this study, the effects of asphaltite and long-term aging on the mechanical properties of hot mix asphalt were investigated. The asphaltite entrained as filler at five different proportions into the hot mix asphalt samples and the obtained results were compared against the control mixtures. The pure and asphaltite containing compacted mixtures were exposed to long-term aging in an oven. Marshall stability, resistance to moisture-induced damage and resistance to crack propagation tests in accordance with the principles of elastic-plastic fracture as well as linear-elastic fracture mechanics were carried out on the mixtures. It was determined that the optimal proportion of asphaltite usage as filler with respect to volumetric design is 3% by weight. The test results demonstrated that asphaltite use enhances the resistance against permanent deformation, moisture-induced damage and crack propagation especially at normal temperatures, albeit at the cost of causing the hot mix asphalt to exhibit a more brittle behavior. The use of asphaltite would be suggested in order to reduce the long-term effects of aging in general specifically on account of the results of the stability, moisture-induced damage and fracture toughness tests.

Keywords: Hot mix asphalt, Mechanical properties, Asphaltite, Long-term aging, Fracture.

Introduction

In the hot mix asphalts (HMA) used in the flexible pavements of highways, various pavement distresses such as permanent deformation, moisture damage and cracks due to low

temperature and fatigue regularly occur. In this context, various modifier materials are employed to enhance the resistance of HMAs against pavement distresses and thus improve the pavement's performance and extend its service life. The modifiers can be added to the bituminous binder as well into the mixture directly. Styrene-butadiene-styrene (SBS) (Al-Hadidy and Yi-qiu 2011; Airey 2003), ethylene-vinyl-acetate (EVA) (Lu et al. 1999; Airey 2002) and rubber (Kok and Colak 2011) were used in the bitumen modification and lime (Kok and Yilmaz 2009) and carbon black (Geckil 2008) were used in mixture modification, thereby the positive impact they had on the variety of pavement distresses encountered in HMAs was observed.

Another type of modifier used in the HMAs is natural hydrocarbons. Natural bitumens are comprised of two subclasses, namely soluble natural bitumens and pyrobitumens. Soluble natural bitumens are then divided into three subclasses as mineral wax, natural asphalt (Athabasca, Trinidad Lake, tabbyite) and asphaltite (gilsonite, grahamite, glance pitch) (Meyer and De Witt 1990). Trinidad Lake asphalt and gilsonite (Liu and Li 2008; Yilmaz et al. 2011) are the most commonly utilized natural hydrocarbons in HMA modification. In Turkey, pylon type natural hydrocarbon deposits at economically feasible thickness are located in the provinces of Sirnak and Silopi in Southeastern Anatolia Region. Although the natural hydrocarbons mined from this region are classified between asphaltite and pyrobitumens with respect to their solubility in carbon disulfide, they are collectively named as asphaltite. In Southeastern Anatolia Region of Turkey, the asphaltite reserves are estimated to total 82 million tons, of which 44.5 million are proved reserves (Kavak 2011). Asphaltite is a complex mixture consisting of compounds ranging from nonpolar aliphatic and naphthenic hydrocarbons to highly polar aromatic molecules containing heteroatoms such as oxygen, nitrogen and sulfide (Bukka et al. 1991). It was previously determined that the sulfur within

asphaltite would increase the performance of HMAs, especially in terms of resistance against permanent deformation (Deniz and Lav, 2010, Timmy *et al.* 2010, Cooper *et al.* 2011).

In bituminous mixtures, aging occurs due to factors like temperature, solar radiation and contact with oxygen. Viscosity of the bituminous binder rises due to aging which also leads to hardening of the HMA. This situation causes the HMAs to display a more brittle structure (Vallerga 1981). The aging process of the bituminous binder is analyzed with respect to both short-term and long-term which depends on the duration of aging. While the short-term aging (STA) takes places during the bitumen storage, overhauling to plant, mixing with the aggregate, hauling to the application area, spreading and compaction stages; the long-term aging (LTA) occurs during the entire service life of the pavement (Whiteoak and Read 2003). Regarding the short-term aging of the mixtures in accordance with Superpave method, it is recommended that the mixtures are exposed to a temperature of 135°C for 4 hours at loose condition prior to compaction (Asphalt Institute 1996).

For the purpose of performing the most realistic simulation under laboratory conditions within the body of SHRP, it was determined from the field and laboratory studies conducted by Bell *et. al.* that keeping the short-term aged mixtures (4 hours at 135°C) in an 85°C-hot air-circulation oven after compaction for 120 hours (5 days) stands out as the most convenient technique for the long-term aging of dense-graded HMAs (Bell *et al.* 1994). The exposure of HMAs to a temperature of 85°C in a hot air-circulation oven for 120 hours (5 days) accurately translates the dynamics of aging for a period of 7-10 years at outdoor conditions into the laboratory environment (Harrigan *et al.* 1994).

Asphaltite was mainly used for heating purposes for a long period in Turkey. Due to its high sulfur content (4-4.5%), its use was recently forbidden in city centers to help curbing excessive air pollution. In this study, asphaltite was used as an additive in hot mix asphalts in order to investigate the utility of this material that possess huge reserves in Turkey in a

different industry. Currently, only a few studies on the application of asphaltite from Turkish mines in hot mix asphalts are available. Moreover, merely a handful of studies were conducted to investigate the impact of asphaltite on the long-term aging of hot mix asphalts. It is envisaged that any positive results obtained from the research may promote the utilization of asphaltite as an additive at both the national and international stage.

Materials and sample preparation

An asphalt cement, PG 64–34, obtained from Turkish Petroleum Refineries was used as binder for the mixture preparation. The mixing and compaction temperatures were determined for the binder by using the 170 ± 20 and 280 ± 30 cP viscosity values, respectively. The rheological properties of bitumen and AASHTO M320 binder specification limits are given in Table 1. Limestone aggregate was used for the asphalt mixtures. The properties of aggregate are given in Table 2. A crushed coarse and fine aggregate with a maximum grain size of 19 mm was chosen for a dense graded asphalt mixture. The gradation of the aggregate mixtures is given in Table 3. The asphaltite was supplied from an asphaltite mine in Silopi region of Turkey. Because of its low hardness value (approximately 2) as per the Mohs scale, the asphaltite was ground and the particles smaller than 0.075 mm were assorted to be used as filler in hot mix asphalt. The physical and the chemical properties of the asphaltite that were used are given in Table 4.

In the study, 6 different types of mixtures were investigated. The filler in the control mixture entirely consisted of limestone, whereas in the test mixtures, asphaltite was used at 5 different proportions ranging from 1% to 5% as filler to partially replaced limestone. The optimum bitumen contents of the mixtures were determined in accordance with Superpave mix design. The volumetric properties and specification limits of the pure and the asphaltite containing mixtures prepared at the optimum levels of bitumen content are presented in Table 5. Accordingly, the optimum bitumen contents dropped as the proportion of asphaltite

increased. Because of bitumen included in asphaltite which is a type of natural asphalt, optimum bitumen content was decreased. As seen in Table 5, the mixtures containing 4% and 5% asphaltite as filler were also subjected to the tests on the hot mix asphalts despite failing to meet the Superpave specification criteria, for the sole purpose of investigating the influence of asphaltite at a wider scope. In addition, the data obtained in the experiments were statistically evaluated through the application of linear regression analysis and the SPSS package software.

Test methods

Marshall stability and flow test

Marshall stability and flow tests were applied according to EN 12697-34 standard test method. The specimens were compacted at $4\pm1\%$ air voids with a Superpave gyratory compactor (SGC). The specimens were divided into four groups each of which consisted of 18 mixtures, and the mean specific gravities of the specimens in each group were equal. The first group was unconditioned (immersed in water at 60°C for 40 min) and short-term aged (S), the second group was conditioned (immersed in water at 60°C for 24 h) and short-term aged (SC), the third group was unconditioned (immersed in water at 60°C for 40 min) and long-term aged (L), and finally the fourth group was conditioned (immersed in water at 60°C for 24 h) and long-term aged (LC). The specimens were loaded to the point of reaching failure by using curved steel loading plates along the diameter at a constant rate of compression of 50.8 mm/min. The stability and flow as well as the ratio of stability (kN) to flow (mm), defined as the Marshall quotient (MQ), which is an indication of stiffness of the mixtures were determined. It is clearly recognized that MQ is a measure of the materials' resistance to shear stress, permanent deformation and hence rutting. High MQ values indicate a mix with high stiffness, greater ability to spread the applied load and resistance to creep deformation

(Zoorob and Suparma 2000). The retained Marshall stability (RMS) was then calculated from the mean stability values of each group by the following formula:

$$RMS = 100 * (MS_{cond.} / MS_{uncond.}) \quad (1)$$

where RMS is the retained Marshall stability (%); $MS_{cond.}$ is the mean Marshall stability for conditioned specimens (kN); and $MS_{uncond.}$ is the mean Marshall stability for unconditioned specimens (kN).

Resistance to moisture-induced damage test

Resistances of the pure and asphaltite including mixtures to moisture-induced damage were determined according to AASHTO T 283 standard test procedure. The specimens were compacted at 7 ± 0.5 air void content with SGC. The specimens were divided into four groups each of which consisted of 18 mixtures, and the mean specific gravities of the specimens in each group were equal. The first group was unconditioned (immersed in water at 25°C for 2 h) and short-term aged (S), the second group was conditioned (in freezer at -18°C for 16 h then immersed in water at 60°C for 24 h and at 25°C for 2 h) and short-term aged (SC), the third group was unconditioned (immersed in water at 25°C for 2 h) and long-term aged (L), and lastly the fourth group was conditioned (in freezer at -18°C for 16 h then immersed in water at 60°C for 24 h and at 25°C for 2 h) and long-term aged (LC). Before the conditioning procedure, the samples were vacuum-saturated so that 70-80% of the encompassed air voids were filled with water. Cylindrical specimens were subjected to compressive loads at a constant rate of 50.8 mm/min, which act parallel to the vertical diametral plane, by using the Marshall loading equipment. Based on the maximum level of load at failure, the indirect tensile strength (ITS) in units of kPa is calculated by the following equation:

$$ITS = 2 * F / \pi * L * D \quad (2)$$

where F is the peak value of the applied vertical load (kN); L is the mean thickness of the test specimen (m); and D is the specimen diameter (m). The indirect tensile strength ratio (TSR) was determined as per the equation below:

$$\text{TSR} = 100 * (\text{ITS}_{\text{cond.}} / \text{ITS}_{\text{uncond.}}) \quad (3)$$

where $\text{ITS}_{\text{cond.}}$ is the indirect tensile strength of the conditioned specimens and $\text{ITS}_{\text{uncond.}}$ denotes the indirect tensile strength of the unconditioned specimens. A TSR value of 0.80 is considered to be the minimum threshold for hot mix asphalts as per Superpave design procedure.

Resistance to crack propagation tests

Hot mix asphalts may exhibit linear-elastic and elastic-plastic behavior depending on the loading speed and temperature. At low temperatures (1°C or less) and convenient loading speeds, HMAs may be assessed in accordance with the principles of linear-elastic fracture mechanics (LEFM), while it might be more suitable to treat them as per elastic-plastic fracture mechanics (EPFM) at normal or high temperatures (25°C and above) (Molenaar 2003).

The fracture toughness (K_{IC}) test as described in EN 12697–44 (2010) standard is carried out for the assessment of resistance against crack propagation in HMAs as per LEFM principles. The specimens were prepared by a gyratory compactor that is 150 mm in diameter and 100 mm thick for the fracture toughness test. The specimens were each sliced into two equal semi-circular pieces. Then the resulting semi-circular specimens were cut into two equal slices each with a thickness of 50 mm. A single notch about 10 mm in depth and 1.5 mm in width was carved in the middle of the specimens. The specimens were loaded at a constant cross-head deformation rate of 5.0 mm/min using a three-point bend loading configuration. The samples were kept at 0°C for 4 hours prior to testing. The load level and the ensuing

deformation were recorded continuously, followed by determination of the fracture toughness (K_{IC} , $N/mm^{3/2}$) and maximum vertical strain (ϵ_{max} , %) using the following equations;

$$\sigma_{max} = \frac{4.263 * F_{max}}{D * t} \quad (4)$$

$$K_{IC} = \sigma_{max} * 5.956 \quad (5)$$

$$\epsilon_{max} = \frac{\Delta W}{W} * 100 \quad (6)$$

where, σ_{max} is the maximum stress at failure (N/mm^2); F_{max} is the maximum force (N); D is the diameter of specimen (mm); t is the thickness of specimen (mm); W is the height of specimen (mm), and ΔW is the deformation at the maximum force (mm).

The critical strain energy release rate approach is employed in the assessment of hot mix asphalt's resistance against crack propagation as per the principles of elastic-plastic fracture mechanics (Mohammad et al. 2004). The critical strain energy release rate, is alternatively called the critical value of J-integral, or J_c in short. In this approach, the specimens were prepared by a gyratory compactor 150 mm in diameter and 100 mm thick. The specimens were each sliced into two equal semi-circular pieces. Then the resulting semi-circular specimens were cut into two equal slices each with a thickness of 50 mm. Three notches at various depths (such as 10, 20 and 30 mm) and 1.5 mm width were carved into the middle of the specimens. The specimens were loaded monotonically at a speed of 0.5 mm/min in a three-point bend load configuration. The tests were conducted at 25°C. The load level and the resulting deformations were recorded continuously and the critical strain energy release rate (J_c , kJ/m^2) was determined using the equation below;

$$J_c = -\left(\frac{1}{b}\right) * \frac{dU}{da} \quad (7)$$

where b is the specimen thickness (mm); U is the strain energy failure values obtained from the load deflection behavior; and a is the notch depth (mm). Crack propagation test configuration and the tested specimens are displayed in Fig. 1.

Results and discussions

Marshall stability and flow test results

The Marshall test results are given in Table 6. The values indicate the means of three samples.

An analysis of the stability values demonstrated that they tend to steadily increase with rising asphaltite content. While stability value of S mixtures jumped by 16% in the presence of 5% asphaltite content, the corresponding rise in SC mixtures was 29%, L mixtures was 13% and LC mixtures was 28%. The increase in stability in S, SC, L and LC mixtures was dependent on the use of asphaltite at ratios of 83%, 93%, 80% and 95%, respectively, as stated in the SPSS results and the constructed models were statistically meaningful. In the formerly conducted studies, it was determined that stability values decreased as the asphaltite content increased in the mixtures prepared as per the Marshall method (Kuloglu et al. 2009; Oruç and Eren 2008). Despite the notable differentials between the aggregate and bitumen characteristics and the respective design methods, it is supposed that this situation arose as a consequence of the distinct chemical structure of the asphaltite used in study since it was supplied at a different time than the materials used in the other studies.

The stability values of the mixtures increased as a result of long-term aging. The aging index (AI_{MS}) was used to assess the influence of long-term aging on the Marshall stability values. AI_{MS} value was obtained by calculating the ratio of the stability values of the long-term aged mixtures to those of the short-term aged mixtures. An aging index value around 1 indicates that the effect of aging on the mixtures was relatively limited.

The Marshall stability ageing index values increased as a result of conditioning. It was found that the AI_{MS} value in all conditioned mixtures was lower than that of the pure mixture

with the exception the mixture containing 3% asphaltite. As for the unconditioned mixtures, the aging index values turned out to be lower compared to the pure mixture in all cases. It can be inferred from the obtained results that the use of asphaltite generally mitigated the effect of aging on the stability of the mixtures.

Analyzing the Marshall quotient values, it was determined that they escalated at higher levels of asphaltite content. The highest MQ values were attained for the S samples containing 3% asphaltite and for the SC, L and LC samples containing 5% asphaltite. The lowest MQ values were attained for the pure samples. This observation demonstrates that asphaltite use enhances the resistance of HMAs against permanent deformation. The boost in the MQ values due to long-term aging indicates that the mixtures will display an even stronger resistance against permanent deformation.

The RMS values increased at higher contents of asphaltite. Among the mixtures, the lowest RMS was displayed by the pure mixture whereas the highest RMS belonged to the mixture containing 5% asphaltite. Using 5% asphaltite in the STA mixtures escalated the RMS value by 12%, while the corresponding increase was 14% in the LTA mixtures. The adjusted R^2 values of the RMS were determined as 0.559 for the STA samples and 0.717 for the LTA samples. Similar to the results obtained on the Marshall stability and MQ values, the RMS values of all mixtures displayed an increase due to LTA. This observation shows that asphaltite use augments the resistance to moisture-induced damage before and after long-term aging.

Resistance to moisture-induced damage test results

The indirect tensile strength test results of mixtures are given in Table 6. The values denote the means of three samples.

It was determined that the highest resistances were displayed by L mixture while the lowest resistance values belonged to SC mixtures. Indirect tensile strengths both before and

after conditioning rose as the asphaltite content increased. In case of entraining 5% asphaltite into S mixtures, the ITS value was higher by 49% relative to the pure mixture, while the corresponding rise was 66% in SC mixtures, 31% in L mixtures and 46% in LC mixtures. The adjusted R^2 values of the ITS were determined as 0.942 for S mixtures, 0.948 for SC mixtures, 0.854 for L mixtures and 0.920 for LC mixtures.

ITS values of the mixtures increased as a result of long-term aging. AI_{ITS} was used to assess the influence of long-term aging on the ITS values of both pure and asphaltite containing mixtures. AI_{ITS} values were obtained by taking the ratio of ITS values of the long-term aged mixtures to those of the short-term aged mixtures.

As seen in Table 6, the aging index rose as a result of conditioning. It was determined that the aging index decreased in both conditioned and unconditioned mixtures as the asphaltite content increased, since the highest (AI_{ITS}) value was displayed by the pure mixture whereas the 5% asphaltite containing mixture yielded the lowest (AI_{ITS}). It can be suggested by analyzing the obtained results that asphaltite use limited the extent of aging's impact on the ITS values.

Analyzing the TSR values which indicate the resistance to moisture-induced damage, it was determined that all of the mixtures surpassed the 80% threshold, which is the lower bound of Superpave standard. Furthermore, the 4% and 5% asphaltite containing mixtures exhibited TSR values above 90%. The adjusted R^2 values of the TSR were determined as 0.278 for the STA samples and 0.236 for the LTA samples. TSR values of the mixtures increased thanks to long-term aging. It can be suggested in view of the RMS and TSR values that asphaltite use will enhance the resistance to moisture-induced damage in HMAs.

Resistance to crack propagation test results

Transgranular fractures were observed in the specimens as a result of the fracture toughness (K_{IC}) tests, whereas J_C tests culminated in intergranular fracture (Fig.2). In (K_{IC}) tests, the

specimens fractured at the moment of reaching the maximum load level, while no disintegration was observed in the specimens even during decompression. The simulated variation of load-deformation obtained from the K_{IC} and J_C tests is depicted in Fig.3.

The fracture toughness and strain at maximum force levels for both STA and LTA mixtures are given in Table 7. Also, aging indices derived from the K_{IC} and ϵ_{mak} values of the mixtures are given in Table 7. The values denote the means of three samples.

It is seen in Table 7 that the highest K_{IC} value among the short-term aged specimens are exhibited by 3% asphaltite containing mixture. As for the long-term aged mixtures, the highest K_{IC} value was measured in the pure mixture, trailed by 3% asphaltite containing mixture. The lowest K_{IC} value belonged to the mixtures containing 5% asphaltite by weight for both STA and LTA mixtures. Higher values of K_{IC} show that the fracture resistance will be high as per the principles of linear-elastic fracture mechanics. The highest fracture resistances at low temperatures (0°C) were demonstrated by the 3% asphaltite containing mixture prior to long-term aging and by the pure mixture after long-term aging. It was determined that the adjusted R^2 values of the mixture K_{IC} 's prior to long-term aging were very low and following LTA, the adjusted R^2 value was 0.536.

A notable decrease in maximum deformation values was observed in the short-term aged specimens after the asphaltite content reached 2% or higher. The pure mixture displayed the highest deformation, whereas the lowest ϵ_{mak} value was yielded by the mixture containing 4% asphaltite by weight. After long-term aging, the maximum deformation value was displayed by the pure mixture while the lowest ϵ_{mak} value was measured in the mixtures containing 1% asphaltite and 5% asphaltite, in increasing order. The adjusted R^2 value was 0.586 for STA mixtures and this value was very low and that the constructed model was statistically insignificant in LTA mixtures. In light of these results, it could be suggested that the highest

elastic deformation until the point of fracturing takes place in pure mixture, whereas the lowest deformation generally occurs in the mixtures with higher levels of asphaltite content.

Fracture toughness increased in all mixtures due to long-term aging while the maximum deformation levels decreased. Table 7 shows that any increase in the asphaltite content accompanied reductions in fracture toughness aging indices (AI_{KIC}). In view of the obtained (AI_{KIC}) values, it was determined that long-term aging had the greatest impact on the pure mixture (a 12.5% rise), whereas having minimal impact on the 4% asphaltite containing mixture (a trivial increase of 0.5%). An analysis of the $AI_{\epsilon_{max}}$ values showed that a significant decrease was observed in the maximum pavement distress values due to long-term aging for asphaltite contents up to 3%, whereas the corresponding decrease was minimal for mixtures containing 3%, 4% and 5% asphaltite by weight. The obtained results suggest that asphaltite use diminishes the extent that the K_{IC} and ϵ_{mak} values are affected by aging.

The maximum load and deformation at maximum load (δ_{max}) values of both STA and LTA mixtures carved with 1, 2 and 3-cm deep notches are given in Fig.4 and Fig.5, respectively. The values denote the means of three samples.

As seen in Figure 4, the maximum load values at all notch depths displayed a steady increase as the asphaltite content increased. Besides, the maximum load levels decreased at higher notch depths, while increasing due to long-term aging. Among the STA mixtures, the largest increase in maximum load relative to the pure mixture was displayed at 80% by the 5% asphaltite containing mixture with a notch depth of 1-cm, while the exact same mixture also had the highest amount of increase among the LTA mixtures at 85%. The correlation coefficient between the maximum load values and the asphaltite content were determined as ca. 0.45 in the statistical evaluation.

As seen from the deformation values at the maximum load level presented in Figure 5, a general decrease in deformation values are observed at rising contents of asphaltite for all

notch depths. Deformation values at maximum loads for all mixtures decreased as a result of long-term aging. The highest $\delta_{\max}$ value among the STA mixtures for all notch depths was obtained from the pure mixture, while the lowest $\delta_{\max}$ value was displayed by the 5% asphaltite containing mixture. As for LTA mixtures, the pure mixture possessed the highest $\delta_{\max}$ value at notch depths of 1-cm and 3-cm, while the 2% asphaltite containing mixture displayed the highest value at 2-cm notch depth. On the other hand, the lowest $\delta_{\max}$ value belonged to the 4% asphaltite containing mixture at 1-cm notch depth and the 5% asphaltite containing mixture at notch depths of 2-cm and 3-cm. The fact that deformation values at maximum load level falls despite the increase in the load at higher asphaltite contents shows that a more brittle fracturing will take place in such cases although the load level that the mixtures can bear increases. Evaluation of the correlation coefficients indicated that it was 0.18 between the $\delta_{\max}$ values and the asphaltite content for the STA mixtures whereas it was 0.20 for the LTA mixtures and that the correlation coefficient between the $\delta_{\max}$ values and the notch depth was 0.83 for the STA mixtures and 0.79 for the LTA mixtures. The variation of area ($U_{\max}$) values with the asphaltite content obtained from the maximum load-deformation curve of the mixtures with differing notch depths is presented in Figure 6.

Analyzing Figure 6, it was determined that the highest and lowest area values among LTA mixtures for all notch depths were displayed by the 4% asphaltite containing mixture and the pure mixture, respectively. Since the area under the maximum load-deformation curve indicates the excess energy stored until the moment of fracturing, it could be inferred that the highest resistance after crack formation and up to the maximum load bearing level will be displayed by the 4% asphaltite containing mixture.

When the long-term aged specimens are analyzed, it was determined that the highest area value at 1-cm notch depth belonged to the 5% asphaltite containing mixture, while the lowest area was yielded by the pure mixture. As for 2-cm notch depth, the area values increased up to

2% asphaltite content and steadily increased afterwards. However, despite this noted drop in area values for contents higher than 2%, the lowest area value at 2-cm notch depth was still possessed by the pure mixture. At 3-cm notch depth, the highest and lowest area values were displayed by the 5% and 1% asphaltite containing mixtures, respectively. The correlation coefficient between the asphaltite content and the $U_{\max}$ values was determined as ca. 0.18. The J_c values were calculated using Formula 7 for the situations when the notch depth was increased from 1cm to 2 cm, from 2 cm to 3 cm and from 1 cm to 3 cm and the values are provided in Table 8.

Analyzing Table 8, it is seen that the lowest J_c value for notch depths in the interval of 1-2 cm for the STA mixtures belonged to the pure mixture, while maximum J_c was displayed by the 3% asphaltite containing mixture (which was 2.04 times that of the pure mixture). When the notch depth interval of 2-3 cm is analyzed, the lowest J_c value was again displayed by the pure mixture, followed by the 1% and 3% asphaltite containing mixtures. The largest J_c was measured in the 2% and 5% asphaltite containing mixtures. Finally, the lowest J_c value in the 1-3 cm notch depth interval was found in the pure mixture, whereas the maximum J_c was displayed by the 5% asphaltite containing mixture.

Analyzing the critical energy release rate (J_c) values in the short-term aged specimens, it was observed that the J_c values obtained from the pure mixture and the mixtures containing asphaltite by 1% and 2% at notch depth intervals of 2-3 cm was higher than the corresponding values for 1-2 cm notch depth interval. However, the opposite situation held true in all other mixtures, which indicates that the resistance against crack propagation in the mixtures containing asphaltite by 3%, 4% and 5% was higher for the notch depth interval of 1-2 cm compared to the 2-3 cm interval. An increase in J_c values was attained through the addition of asphaltite as shown by the investigation of the 1-3 cm crack intervals. This demonstrated the

increase in the resistance of bitumen containing hot mix asphalt samples against crack propagation through the use of asphaltite.

In LTA mixtures, the lowest J_C value for 1-2 cm notch depth interval was displayed by the pure mixture, while the highest value belonged to the 5% asphaltite containing mixture (whose J_C value was 2.51 times that of the pure mixture). For the 2-3 cm interval, the lowest J_C value was likewise exhibited by the pure mixture, while the J_C value rose up to an asphaltite content of 2% and decreased beyond this level. Analyzing the results for the 1-3 cm interval, it was determined that the lowest J_C value was again possessed by the pure mixture, while the highest value belonged to the 5% asphaltite containing mixture, whose J_C value was equal to 2.1 times the pure mixture's value. The correlation coefficient between the asphaltite content and the J_C values was determined as 0.611 for the STA mixtures and as 0.278 for the LTA mixtures.

Analyzing the critical energy release rate (J_C) values in the long-term aged specimens, it was observed that the J_C values of all mixtures at notch depth interval of 1-2 cm was higher than the corresponding values for 2-3 cm notch depth interval. These results indicate to a stronger resistance against crack propagation after long-term aging in all mixtures for 1-2 cm notch depth interval compared to the resistance in case of 2-3 cm interval. The results for the 1-3 cm interval indicate to a rise in J_C values with increasing levels of asphaltite content, which demonstrate that resistance against crack propagation will be enhanced in hot mix asphalts as the asphaltite content increases, in compliance with the observations on short-term aged specimens.

The aging indices constructed from the J-integral tests conducted in accordance with the elastic-plastic fracture mechanics approach were calculated by taking the ratio of the values measures for the long-term aged specimens to those measured for the short-term aged specimens. The obtained aging indices are shown in Table 9.

Irregular changes were determined in the behavior of maximum load aging indices corresponding to increases in asphaltite content. In view of the table 9, the maximum aging index values were observed in the case of 3-cm notch depth.

Analyzing the variation in $\delta_{\max}$ aging indices with the asphaltite content, it was determined that the highest aging index value was displayed by the 5%, 3% and again 5% asphaltite containing mixtures at notch depths of 1-cm, 2-cm and 3-cm, respectively. On the other hand, the lowest index values were displayed by the 4% asphaltite containing mixture for notch depths of 1-cm and 2-cm, and by the 1% mixture for 3-cm notch depth. The higher deformation indices displayed by the 3% and 5% asphaltite containing mixtures were attributed to the proximity of deformation values subsequent to long-term aging and the corresponding values in the wake of short-term aging, in addition to the low level of deformation in short-term aged specimens. When the variations in aging indices pertaining to J_c values with the asphaltite content were analyzed, it was seen that some values remained below 1 while others exceeded this threshold.

Conclusions

In this study, laboratory performance tests were performed on both short-term and long-term aged HMA mixtures with different asphaltite contents. Based on the results and analyses from this study, the relevant findings and conclusions can be summarized as follows:

The bitumen demand of the mixtures decreased as the asphaltite content increased. The 4% and 5% asphaltite containing mixtures failed to satisfy the VMA specification criteria as per Superpave method. Hence, the most convenient level of asphaltite content to be used in the filler was determined as 3%.

As a result of Marshall stability and flow tests, it was found that the stability values increased as the asphaltite content increased. Moreover, the stability values of the mixtures also rose due to long-term aging. Increases in AI_{MS} values due to conditioning were identified.

Collectively, it can be suggested in view of the results that asphaltite use mitigates the impact of aging on the stability values.

The MQ values rose at higher contents of asphaltite. This situation indicates that the resistance of HMAs against permanent deformation could be enhanced by asphaltite use. The fact that MQ values of the mixtures increased due to LTA suggests that they will exhibit higher resistance against permanent deformation after long-term aging.

The indirect tensile strength values increased with rising levels of asphaltite content both prior to and subsequent to conditioning. It was determined that the aging indices decreased in both conditioned and unconditioned mixtures as the asphaltite content increases, in addition to observing the lowest and highest AI_{ITS} values in the pure mixture and the 5% asphaltite containing mixture, respectively. The obtained results suggest that the use of asphaltite mitigates the effect of aging on the ITS values.

RMS and TSR values climbed steadily as the asphaltite content increased. Besides, RMS and TSR values in all mixtures escalated due to LTA. The obtained results demonstrate that the resistance of asphaltite containing mixtures against moisture-induced damage is greater both in advance of and in the wake of long-term aging.

A general decrease in ϵ_{mak} value was determined with rising asphaltite content. Assessing the K_{IC} values, it could be suggested that the highest fracture strength at a low temperature level (0°C) prior to long-term aging is displayed by the 3% asphaltite containing mixture whereas the lowest strength is displayed by the 5% asphaltite containing mixture. As for the fracture strength subsequent to long-term aging at low temperature (0°C), the highest value belong to the pure mixture, while the lowest strength was again exhibited by the 5% asphaltite containing mixture. As the fracture toughness values of all mixtures rose due to long-term aging, the maximum deformation levels decreased. It was determined that the aging indices

pertaining to fracture toughness (AI_{KIC}) decreased as the asphaltite content increased, indicating the diminishing effect of aging.

The maximum load levels at all notch depths increased in line with asphaltite content, whereas deformation at maximum load (δ_{max}) values decreased. It was determined from the area (U_{max}) values falling under the maximum load-deformation curve that the highest area among LTA mixtures belonged to the 4% asphaltite containing mixture, while the pure mixture yielded the lowest area. Among the long-term aged mixtures, the highest area was generally displayed by the mixture containing the higher proportion of asphaltite.

The J_C values of all asphaltite containing mixtures turned out to be higher than the pure mixture, which indicates that the resistance against crack propagation in hot mix asphalts should be augmented by increasing the asphaltite content. When the aging indices of J_C values were analyzed, it was seen that asphaltite use had no regular influence on aging.

After a comprehensive evaluation of all test results and observations, it was determined that asphaltite use presumably reduces bitumen requirement, enhances the strength against permanent deformation, and resistance to moisture-induced damage and crack propagation, albeit at the cost of inducing a more brittle behavior on the HMA. As a result, it could be deduced that the use of asphaltite would generally reduce the effects of long-term aging as indicated by the results of the stability, moisture-induced damage and fracture toughness tests. The conducted economical evaluation indicated that the increase in costs resulting from the addition of asphaltite balanced out with the decrease in costs resulting from the decrease in the overall bitumen content in design. Due to the fact that asphaltite causes hot mix asphalts to become more brittle, it would be suggested to investigate the effect of this material on the low temperature crack resistance of HMAs more thoroughly. In addition, the full-scale investigation of the effect of traffic and environmental conditions on the asphaltite containing mixtures through land applications presents utmost significance for the study to gain validity.

Acknowledgements

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Figure Captions

Fig. 1. LEFM and EPFM test configurations and tested specimens.

Fig. 2. Intergranular (left) and transgranular (right) cracking occurred in samples.

Fig. 3. Load-deformation relationship for fracture toughness (left) and J-integral tests (right) at 1 cm notch depth.

Fig. 4. Maximum load values of mixtures.

Fig. 5. Deformation at maximum load values of mixtures.

Fig. 6. $U_{\max}$ values of mixtures.

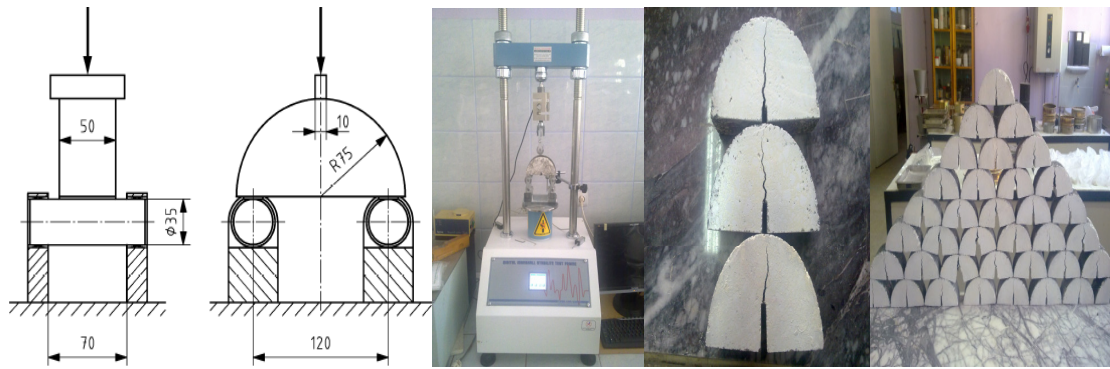


Fig. 1.

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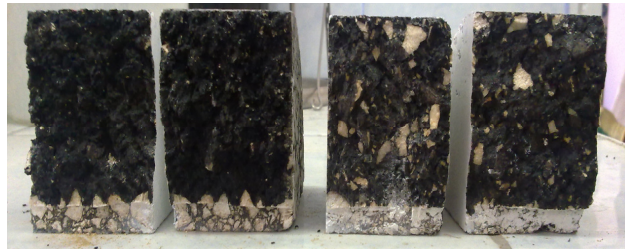


Fig. 2.

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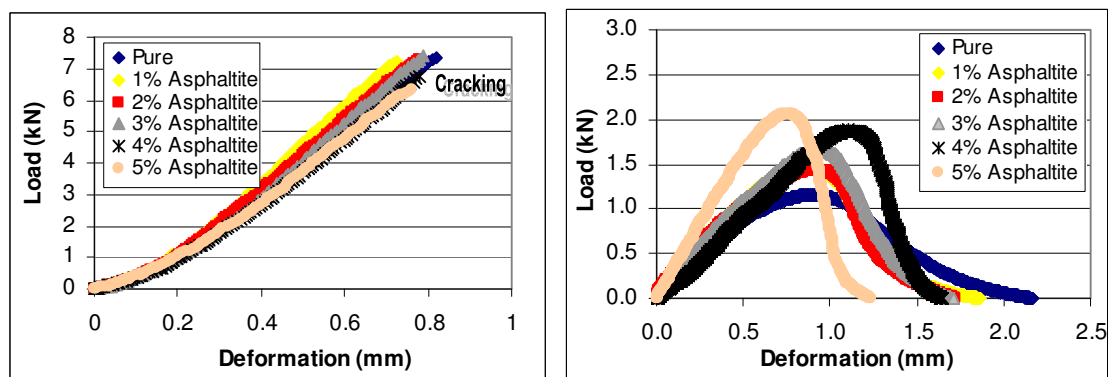


Fig. 3.

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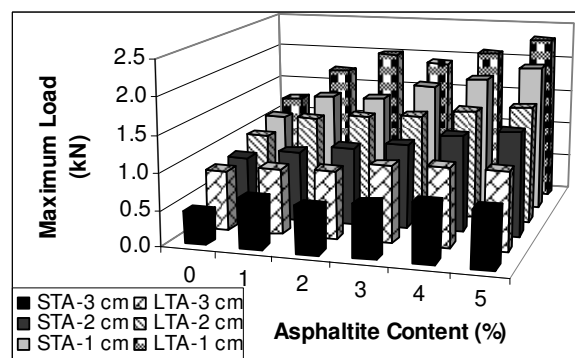


Fig. 4.

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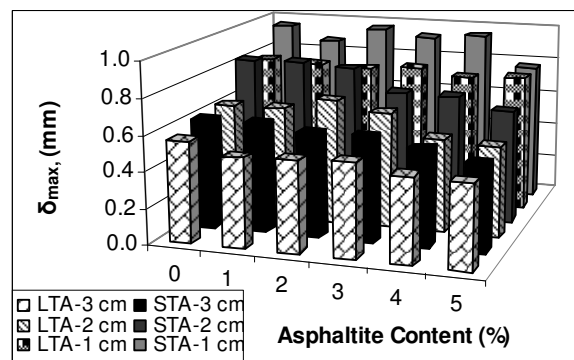


Fig. 5.

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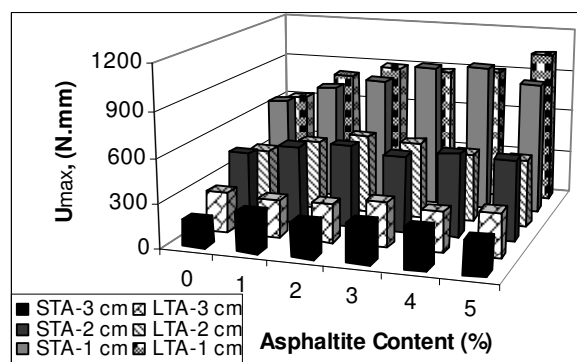


Fig. 6.

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Table 1

Fundamental properties of PG 64-34 binder.

Properties	Binder type	Results	Specification limits
Viscosity (cP, 135°C)	Original	500	max. 3000
Viscosity (cP, 165°C)	Original	162.5	–
$G^*/\sin\delta$ (kPa), 64°C	Original	1.50	min. 1.0
Mixing temperature range (°C)	Original	159-165	–
Compaction temperature range (°C)	Original	145-151	–
Mass loss (%)	RTFOT Residue	0.769	max. 1.0
$G^*/\sin\delta$ (kPa), 64°C	RTFOT Residue	5.78	min. 2.2
$G^{**}\sin\delta$ (kPa), 19°C	PAV Residue	4.58	min. 5000
Creep stiffness (MPa), –24°C	PAV Residue	225.6	max. 300
m-value, –24°C	PAV Residue	0.3047	min. 0.300

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Table 2

Physical properties of aggregate.

Properties	Standard	Aggregate			
		Coarse	Fine	Filler	Asphaltite
Abrasion loss (%)	ASTM	29	–	–	–
(Los Angeles)	DC131				
Frost action (%)	ASTM	4.5	–	–	–
(with Na ₂ SO ₄)	C88				
Coarse aggregate	ASTM	98/96	–	–	–
angularity (%)	D5821				
Fine aggregate	AASHTO	–	49	–	–
angularity (%)	T304				
Flat, elongated	ASTM	2	–	–	–
particles (%)	D4791				
Specific gravity (g/cm ³)	ASTM C127	2.675	–	–	–
Specific gravity (g/cm ³)	ASTM C128	–	2.687	–	–
Specific gravity (g/cm ³)	ASTM D854	–	–	2.711	1.483

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Table 3

Combined aggregate gradation.

Sieve size	Total cumulative passing (%)
19 mm (3/4")	100
12.5 mm (1/2")	95
9.5 mm (3/8")	88
4.75 mm (# 4)	65
2.36 mm (# 8)	39
1.18 mm (# 16)	24
0.600 mm (# 30)	18
0.300 mm (# 50)	14
0.150 mm (# 100)	9.5
0.075 mm (# 200)	5

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Table 4

Physical and chemical properties of asphaltite (Kavak, 2011)

Properties	results
Penetration (0.1 mm), 100 g, 5 s	0
Softening point (°C)	-
Ash content (%)	32.53
Moisture content (%)	6.14
Specific gravity	1.48
Carbon (weight %)	54.22
Hydrogen (weight %)	5.07
Nitrogen + Oxygen (weight %)	0.25

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Table 5

Volumetric properties of pure and asphaltite modified mixtures.

Mixture properties	Specification	Asphaltite content (%)					
	limits	0	1	2	3	4	5
Optimum binder content (%)	–	5.27	5.19	5.15	5.10	5.08	5.04
Volume of air voids (Va, %)	4.0	4.00	4.01	4.05	3.98	3.98	4.03
Voids in the mineral aggregate (VMA, %)	min. 14.0	15.44	15.00	14.52	14.01	13.38	13.10
Voids filled with asphalt (VFA, %)	65–75	74.12	73.27	72.15	71.58	70.27	69.21
Dust proportion (DP)	0.8–1.6	0.98	1.02	1.07	1.11	1.18	1.22
%Gmm@Nini. = 8 (%)	max. 89	85.56	85.59	84.97	85.28	85.64	85.56
%Gmm@Ndes. = 100 (%)	96	96.00	95.99	95.95	96.02	96.02	95.97
%Gmm@Ndes. = 160 (%)	max. 98	96.55	97.29	97.11	97.30	97.77	97.52

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Table 6

Marshall and ITS test results.

		Asphaltite content (%)					
		0	1	2	3	4	5
Marshall test results							
Stability (kN)	S	17.54	18.26	19.22	19.30	19.11	20.33
	SC	14.04	14.75	15.63	15.83	16.94	18.15
	L	19.10	19.83	19.79	20.89	20.44	21.54
	LC	15.41	16.16	16.57	17.87	18.41	19.78
AI _{MS}	Uncond.	1.089	1.086	1.030	1.083	1.070	1.060
	Cond.	1.098	1.097	1.061	1.130	1.087	1.090
MQ (kN/mm)	S	4.79	5.11	5.16	5.88	5.34	5.50
	SC	3.57	3.80	4.08	4.64	4.66	4.76
	L	5.56	5.66	5.85	6.46	6.13	6.64
	LC	4.02	4.55	4.65	5.35	5.36	5.69
RMS (%)	STA	80.0	80.8	81.3	82.0	88.6	89.3
	LTA	80.7	81.5	83.8	85.6	90.1	91.8
ITS test results							
ITS (kPa)	S	840	936	1061	1114	1198	1254
	SC	702	797	922	988	1075	1163
	L	996	1042	1159	1218	1253	1301
	LC	843	908	1021	1086	1141	1226
AI _{ITS}	Uncond.	1.18	1.11	1.09	1.09	1.05	1.04
	Cond.	1.20	1.14	1.11	1.10	1.06	1.05
TSR (%)	STA	83.5	85.1	86.9	88.7	89.8	92.7
	LTA	84.7	87.1	88.1	89.1	91.0	94.3

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Table 7

Fracture toughness (K_{IC}) test results.

		Asphaltite content (%)					
		0	1	2	3	4	5
K_{IC} (N/mm ^{3/2})	STA	23.3	23.7	23.3	25.3	23.1	21.2
	LTA	26.3	25.6	24.8	26.0	23.2	21.5
AI_{KIC}		1.125	1.080	1.062	1.026	1.005	1.015
ϵ_{max} (%)	STA	1.753	1.535	1.735	1.249	1.159	1.265
	LTA	1.191	1.049	1.138	1.138	1.140	1.092
$AI_{\epsilon_{max}}$		0.680	0.684	0.656	0.911	0.983	0.863

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Table 8

J_C values for different notch depths.

Asphaltite content (%)	J_C (kJ/m ²)					
	STA			LTA		
	1–2 cm	2–3 cm	1–3 cm	1–2 cm	2–3 cm	1–3 cm
0	0.434	0.532	0.483	0.458	0.243	0.351
1	0.494	0.530	0.512	0.645	0.466	0.556
2	0.603	0.662	0.632	0.700	0.591	0.646
3	0.886	0.496	0.691	0.720	0.449	0.584
4	0.805	0.557	0.681	0.919	0.371	0.645
5	0.723	0.668	0.695	1.152	0.304	0.728

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Table 9

Aging indexes of mixtures obtained from J_C test results.

Asphaltite content (%)	Aging index											
	Maximum load			$\delta_{\max}$			$U_{\max}$			J_C		
	1 cm	2 cm	3 cm	1 cm	2 cm	3 cm	1 cm	2 cm	3 cm	1-2 cm	2-3 cm	1-3 cm
0	1.11	1.18	1.83	0.83	0.76	0.92	0.94	0.85	1.51	1.06	0.46	0.73
1	1.18	1.33	1.34	0.89	0.75	0.82	1.03	0.90	0.94	1.31	0.88	1.09
2	1.36	1.26	1.45	0.80	0.85	0.87	1.03	0.97	1.13	1.16	0.89	1.02
3	1.12	1.21	1.44	0.86	0.93	0.89	0.90	1.01	1.12	0.81	0.91	0.85
4	1.15	1.14	1.37	0.80	0.75	0.88	0.91	0.84	1.02	1.14	0.67	0.95
5	1.14	1.13	1.40	1.00	0.80	0.96	1.16	0.84	1.33	1.59	0.46	1.05

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