Influence of Temperature on Mechanical Properties of Asphalt Concrete Mixture

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Abstract. Asphalt binder is a thermoplastic material that conducts as an elastic solid at lower service temperatures or throughout fast loading rate. At a high temperature or slow rate of loading, asphalt binder conducts as a different liquid. The classical duplication generates a required to assess the mechanical properties of asphalt concrete at the anticipated service temperature to reduce the stress cracking, which happens at lower temperatures, fatigue, and the plastic deformation at higher temperatures (rutting). In this study, an achievement was made to assess the effect of temperature on the mechanical characteristics of asphalt concrete mixes. A total of 132 asphalt concrete samples were attended utilizing two asphalt cement grades (40-50) and (60-70), and one aggregate gradation (type III A for wearing course) SCRB (R/9, 2003). The specimens were then tested at five different temperatures represented by 5, 15, 25, 40, and 60°C to estimate their mechanical characteristics, including resilient modulus (Mr), permanent deformation, and fatigue features as Marshall features. The average resilient modulus (Mr), which belongs to a temperature of 5°C, was 328036 psi revealing an approximate loss of 88% of its strength in resilient modulus when there is an increase in temperature over 60°C. Meanwhile, there is an increase in the permanent deformation accumulation rate (slope value) of about three folds as the temperature changes from (5-60) °C whereas the fatigue life reduces 32 % with the rise in temperature from (5-25) °C.

Keywords: Asphalt concrete; temperature; mechanical properties; rutting; fatigue.

1. Introduction
Hot mix asphalt concrete (HMA) consists of two substances: aggregate and asphalt binder. A percentage of 94 to 96 by weight of the mix includes the aggregate, while (4–6) percent by weight of the mix consists of the asphalt binder. As much as the percent of the asphalt binder is reasonably small, it dramatically influences the performance of pavement due to the environmental factor represented by temperature affects the asphalt binder more than aggregate [1]. Due to the thermoplastic nature of asphaltic material, the mechanical behavior of asphalt concrete is highly dependent upon the temperature. At high temperatures for a given loading rate, the hot mix asphalt concrete is less stiff than that in low temperature, and hence it is less likely to protect the base and the subgrade from being overstressed adequately. Within the hot mix asphalt concrete layer itself, the resistance to permanent deformation drastically decreases at high temperatures. While at low temperatures, the hot mix asphalt concrete is stiffer and coupled with thermally-induced volume concentration. Consequently, relatively high tensile stresses are mobilized, leading to a great potential for thermal cracking [2].

In the last decade, the road pavement in Baghdad and the other cities in Iraq have shown earlier failure, which negatively impacts the safety of roadway and construction economy. Frequently, the related modes of failure represented by fatigue and rutting are the essential failure kinds that existed in
those newly constructed asphalt concrete pavements. The investigations show that the main reasons of these failures have been carried out and showed that the failure could be classified into extrinsic and intrinsic. The first type can be caused by the heavy axle loading combined with high temperatures (60°C retained for about three months in the pavement). The second category is specified to the mix itself [3]. According to the evolution of the 1986 AASHTO guide, it has been recognized that the future design steps of flexible pavement structures would be based on Mechanistic-based approach principles [4]. In the last 20 years ago, road agencies have tended to direct their attempts towards a Mechanistic-based approach; probably this was the reason behind the replacement of the AASHTO 1993 empirical guide method with the more credible mechanistic-empirical (M-E) design method in 2004. The design method of mechanistic-empirical shares the elements of mechanistic modeling and performance observation in identification the requested pavement thickness for a set of design conditions.

The mechanical modeling is based on preliminary physics and limits pavement reactions to wheel load in terms of stress, strain, and displacement. On the other hand, the impractical section of the design includes the use of the pavement reactions to forecast the life of the pavement on the basis of laboratory-derived models. In both parts of this design procedure (mechanical and empirical), the temperature plays a significant role in the output results. In terms of the mechanical part, the calculation of stresses and strain in each layer of the flexible pavement structure is dependent upon the temperature that governs the elastic modulus of the asphaltic layers. Therefore, the selected air or, in turn, the pavement temperature can duplicate the stress value that could reach the top of the subgrade layer from the axle loading due to the deterioration in the resilient modulus value of asphaltic layers at high temperature. Regarding the empirical part of the design procedure, the temperature significantly influences the performance in terms of rutting and fatigue modes of distress [5]. The mechanical parameters for these two types of distress depend upon the temperatures at which they were characterized in the laboratory. Therefore, it is very obvious that accurate knowledge of temperature effects should be available to allow realistic analyses and design of flexible pavement structures using the mechanistic-empirical design procedure.

2. Material characterization

In this work, aggregate, filler, and asphalt cement were used and described utilizing routine tests. Then, the consequence was contrasted with the Iraqi State Corporation for Roads and Bridges specifications [6].

2.1 Asphalt cement

The two bituminous types utilized in this work are PG 70-16 and PG 64-16, which have been gained from the Dora refinery in Baghdad. Table 1 shows the properties of asphalt.

| Test | PG 70-16 | PG 64-16 |
|------|----------|----------|
| Test Temp. (°C) | Value | Test Temp. (°C) | Value |
| G*/sin*(original) | 70 | 1.0619 kPa | 64 | 1.0603 kPa |
| G*/sin*(RTFO) | 70 | 2.6558 kPa | 64 | 2.6602 kPa |
| G*/sin*(RTFO-PAV) | 31 | 3100 kPa | 28 | 4270 kPa |
| Stiffness, S (RTFO-PAV) | -6 | 117 MPa | -6 | 122.5 MPa |
| Slope, m (RTFO-PAV) | -6 | 0.309 | -6 | 0.335 |

2.2 Aggregate

In this study, the crushed quartz aggregate type from Al-Nibaie quarry has been used that is commonly utilized for bituminous mixtures in Baghdad. This type of aggregate was sieved and combined in a suitable ratio to meet the surface course gradation type III A as requested by SCRB specification [6].
Table 2 indicates the physical properties and aggregate gradation with a nominal maximum size of 12.5 mm (1/2 inch). The gradation with specification limits is shown in Figure 1.

### Table 2. Physical properties of aggregates.

| No. | Laboratory Test          | ASTM Designation | Test Results | SCRIB Specification |
|-----|--------------------------|------------------|--------------|---------------------|
|     | 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, %      |                  | -            | 96                  |
| 5   | Percent Wear (Los Angeles Abrasion), % | C-131 | 17.5 | 30 Max. |
| 6   | Soundness Loss by Magnesium Sulfate solution, % | C-88 | 3.83 | 18 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   | Sand Equivalent, %       | D-2419           | 68.45        | 45 Min.             |

2.3 Filler
The mineral filler is a non-plastic substance that passing sieve number 200 (0.075mm). Limestone dust is used in this work in which it has been obtained from the Mayoralty of Baghdad asphalt concrete mix plant. The source of this filler is the lime factory in Karbala Governorate. Table 3 below shows the physical properties of it.

### Table 3. Physical characteristics of mineral filler.

| Property                        | Value |
|---------------------------------|-------|
| Specific Gravity                | 2.72  |
| % Passing Sieve No.200 (0.075 mm)| 96    |
3. Experimental work

The experimental work has been carried out firstly by identifying the optimum asphalt content (OAC) for all asphalt concrete mixes employing the Marshall mix design method. Secondly, the asphalt concrete mixes were prepared at their (OAC) optimum plus 0.5% and optimum minus 0.5%. Then, they have been tested to assess the mechanical features, which consist of the resilient modulus, permanent deformation, and fatigue features. These characteristics have been obtained utilizing uniaxial repeated loading at five testing temperatures (5, 15, 25, 40, and 60°C) and repeated flexural beam tests at three testing temperatures (5, 15, and 25°C).

3.1 Marshall mix design

According to (ASTM D-1559) specifications [7], the Standard method of Marshall has been used in this study to obtain the optimum bitumen (asphalt cement) content for the asphalt concrete mixtures. The test specimen geometry is 6 inches (101.6 mm) in diameter and 2.5 inches (63.5 mm) in height. The average bitumen content that belongs to a maximum unit, maximum stability, and 4 percent air void was calculated to determine the optimum content for bitumen.

3.2 Uniaxial repeated loading test

According to the pneumatic repeated load system (see Figure 2), the uniaxial repeated loading tests were carried out for cylindrical samples. The tested samples were 101.6 mm (4 inches) in diameter and 203.2 mm (8 inches) in height. A repetitive compressive loading with a stress level of 20 psi was implemented in the form of the rectangular wave with a fixed loading frequency of 1 Hz (0.1 sec. load period and 0.9 sec. rest interval). Accordingly, the permanent axial deformation was calculated at the various loading repetitions noting that the tests were performed at five temperatures (5, 15, 25, 40, and 60°C). The following equation has been utilized to calculate the permanent strain ($\varepsilon_p$):

$$\varepsilon_p = \frac{P_d \times 10^6}{h}$$  \hspace{1cm} (1)

Where $\varepsilon_p$= axial permanent microstrain, $P_d$= axial permanent deformation, and $h$= specimen height.

In this test, the resilient deflection is also measured at the load repetition of 50 to 100, and the resilient strain ($\varepsilon_r$) and resilient modulus (MR) are calculated as follows:

$$\varepsilon_r = \frac{r_d \times 10^6}{h}$$  \hspace{1cm} (2)

$$MR = \frac{\sigma}{\varepsilon_r}$$  \hspace{1cm} (3)

Where $\varepsilon_r$= axial resilient micro strain, $r_d$= axial resilient deflection, $h$= specimen height, MR= Resilient modulus, and $\sigma$ = repeated axial stress.

Equation 4 which has been suggested by [8] is used in this study to calculate the permanent deformation using the linear log-log relationship between permanent micro strain and the number of load repetitions the as follow:

$$\varepsilon_p = aN^b$$  \hspace{1cm} (4)

Where $N$=number of stress applications, $a$= intercept coefficient, and $b$= slope coefficient.
3.3 Flexural beam fatigue test
To estimate the fatigue performance of asphalt concrete mixes, a four-point flexural fatigue bending test was adopted in this study using the pneumatic repeated load system. The flexural test has been carried out in a stress-controlled mode. The applied stress level was varying between five to 30% of the ultimate indirect tensile strength with a frequency of 2 Hz and 0.1 s loading and 0.4 s unloading durations and in a rectangular waveform shape. The test has been conducted at three different temperatures (5, 15, and 25°C) on beam specimens with dimensions 76 mm (3 in) × 76 mm (3 in) × 381 mm (15 in) (as shown below in Figure 3). These beam specimens were equipped as to the method indicated in [9]. For each fatigue test, the initial tensile strain has been calculated at the 50th repetition utilizing Eq. 5. Then, the initial strain has been plotted against the number of repetitions. Destroy of the beam was identified as a failure, and a straight line can approach the plot according to Eq. 6.

\[
\varepsilon_t = \frac{\sigma}{E_s} = \frac{12h\Delta}{3L^2 - 4a^2}
\]

\[
N_f = K_1(\varepsilon_t)^{-K_2}
\]

Where \(\varepsilon_t\) = Initial tensile strain, \(\sigma\) = Extreme flexural stress, \(E_s\) = Stiffness modulus based on center deflection, \(h\) = Height of the beam, \(\Delta\) = Dynamic deflection at the center of the beam, \(L\) = Length of the span between supports, \(a\) = Distance from support to the load point (L/3), \(N_f\) = Number of repetitions to failure, \(K_1\) = fatigue constant, the value of \(N_f\) when \(\varepsilon_t = 1\), and \(K_2\) = inverse slope of the straight line in the logarithmic relationship.
4. Results and discussion

4.1 Marshall properties

Marshall mix design steps, according to [10] have been performed in this study utilizing 75 blows of the automatic Marshall compactor on each face of the sample. This manner can simulate the pressure applied by the tire on the roadway. 15 Marshall samples were equipped for each kind of asphalt cement, with a fixed increase of 0.3% of bitumen content (3 replications for each content). Five different asphalt contents, belong to the mix type IIIA of wearing course (with NMA12.5 mm), have been using to apply the Marshall mix design. These values were 4.3, 4.6, 4.9, 5.2, and 5.5% by weight of the total mixture. The plots of Marshall data for each type of asphalt cement have been shown in Figures 4 and 5. According to these Figures, the determined (OAC) is 5% for bitumen with PG (70-16) and 4.6 percent for PG (64-16).

Figure 4. Marshall plots for mixes with PG (70-16).
4.2 Effect of temperature on resilient modulus

Table 4 and Figure 6 appear the variation of the resilient modulus (Mr) values with the temperatures. Figure 7 shows that the Mr values are substantially affected by the temperature regardless of the other variables (asphalt cement type and content). The higher the temperature, the lower the resilient modulus. According to the data presented in Table 4, the resilient modulus corresponding to temperature 5°C is approximately 1.9 and 8.8 times that corresponding to 25°C and 60°C, respectively. If linear regression is assumed between the resilient modulus value and temperature, the constant of proportionality will be approximately -5100 psi per 1 degree centigrade. This finding does not meaningfully affect the type of asphalt or content that is considered in this study.

Figure 5. Marshall plots for mixes with PG (64-16).
Figure 6. Temperature effect on resilient modulus.

Table 4. Resilient modulus results.

| Temp. (°C) | AC type | 5      | 15     | 25     | 40     | 60     |
|------------|---------|--------|--------|--------|--------|--------|
|   AC (%)   |         |        |        |        |        |        |
| opt-0.5    | PG70-16 | 376471 | 290909 | 213333 | 106667 | 45714  |
|            | PG64-16 | 320000 | 250000 | 160000 | 91429  | 35556  |
| opt        | PG70-16 | 355556 | 278261 | 177778 | 100000 | 44444  |
|            | PG64-16 | 313333 | 242973 | 152381 | 85333  | 32000  |
| opt+0.5    | PG70-16 | 320000 | 266667 | 164103 | 80000  | 35556  |
|            | PG64-16 | 282857 | 208421 | 142222 | 64000  | 29091  |

4.3 Effect of Temperature on Permanent Deformation Parameters
The results of permanent deformation tests in terms of the parameters intercept (a) and slope (b) are shown in Figures 7 and 8, respectively, which is based on the data shown in Tables 5 and 6. Examinations of the appeared data suggested that the permanent deformation parameters intercept and the slope are commonly increased with temperature increase. This finding confirms that the rutting mode of failure is enhanced in asphalt concrete pavement in hot summer temperatures. The slope value that reflects the cumulation rate of permanent deformation for 60 °C is nearly three folds higher than that at 5°C. At the same time, the corresponding value of the intercept coefficient is approximately 268 percent. The average constant of proportionality for the relation between the temperature and slope is + 0.88 percent per 1-degree cent grade. This constant is affected by asphalt type as well as content, as the asphalt cement becomes softer (i.e., PG 64-16). The constant increase to approximately +0.90 percent per 1 degree centigrade, whereas when the asphalt content increased 0.5 percent (by weight of total mix) beyond the optimum content, the constant proportionality increased approximately 6 percent. It can also be noticed from Figure 7 and Table 5 that the intercept values of the ‘opt+’ mixtures are usually higher than that of the ‘opt-’ mixtures, especially for the high temperatures. This is because of the lack of aggregate particles bonding and interacting to each other in the ‘opt-’ mixtures.
Figure 7. Temperature effect on intercept parameter.

Table 5. Intercept results.

| Temp. (°C) | AC (%) | AC type | Intercept (a), Micro strain |
|------------|--------|---------|----------------------------|
| 5          | AC type | opt-    | PG70-16 115                |
|            |        | PG64-16 | 102                  |
|            |        | PG70-16 | 84                   |
|            |        | PG64-16 | 84                   |
|            |        | PG70-16 | opt+ 67                |
|            |        | PG64-16 | 68                   |

Figure 8. Temperature effect on slope parameter.
Table 6. Slope results.

| Temp. (°C) | AC. (%) | AC type   | Slope (b) |
|------------|---------|-----------|-----------|
|            |         | PG70-16   | 0.162     | 0.286     | 0.365     | 0.514     | 0.648     |
| opt-       |         | PG64-16   | 0.178     | 0.311     | 0.372     | 0.529     | 0.687     |
| opt        |         | PG70-16   | 0.224     | 0.293     | 0.374     | 0.551     | 0.671     |
| opt+       |         | PG64-16   | 0.27      | 0.321     | 0.391     | 0.56      | 0.752     |
|            |         | PG70-16   | 0.232     | 0.297     | 0.387     | 0.566     | 0.733     |
|            |         | PG64-16   | 0.273     | 0.317     | 0.402     | 0.583     | 0.758     |

4.4 Effects of Temperature on Fatigue Performance

The fatigue parameters K1 and K2 appear in Figures 9 and 10, respectively, based on the data that appeared in Tables 7 and 8. The values of fatigue parameters (K1 and K2) can indicate the impacts of temperature on the fatigue properties of a paving mix. The flatter the slope of the fatigue curve means the more significant amount of K2. If two substances have the same K1 value, then a large amount of K2 refers to a probable for longer fatigue life. However, a lower amount of K1 represents a shorter fatigue life when the fatigue curves are parallel. That is, K2 is fixed. The consequences show that as the temperature rise from 5°C to 25°C, the K1 value increased while the K2 value decreased. This means a contradiction in behavior between K1 and K2 against temperature. The fatigue life results shown in Figure 11 and presented in Table 9 obviously show that fatigue life increased as the temperatures decrease. Numerically, the fatigue life approximately decreases 32% with a rise in temperature from 5°C to 25°C. Regarding the effect of asphalt type on the K2 parameter, it can be seen from the data shown in Table 8 that the softer asphalt cement PG 64-16 perform better (higher K2 value) against fatigue failure than the hard grade asphalt cement PG 70-16 at all testing temperature except 25°C with optimum -0.5 percent asphalt cement content. With regards to the asphalt cement content, their effect on the K1 value is more pronounced than their effect on K2 value.

Figure 9. Temperature effect on fatigue parameter (K1).
Table 7. Fatigue parameter K1 results.

| Temp. (°C) | AC. (%) | AC type | 5    | 15    | 25    |
|------------|---------|---------|------|-------|-------|
|            |         |         | K1   |       |       |
| opt-       | PG70-16 | 1.64E-07| 3.1E-06| 2.87E-05|
|            | PG64-16 | 1.38E-09| 2.82E-06| 1.14E-05|
| opt        | PG70-16 | 1.52E-09| 4.32E-07| 1.47E-05|
|            | PG64-16 | 4.02E-09| 4.94E-08| 1.85E-06|
| opt+       | PG70-16 | 5.8E-09  | 1.39E-08| 3.26E-08|
|            | PG64-16 | 1.17E-09| 4.77E-09| 8.36E-09|

Figure 10. Temperature effect on fatigue parameter (K2).

Table 8. Fatigue parameter K2 results.

| Temp. (°C) | AC. (%) | AC type | 5    | 15    | 25    |
|------------|---------|---------|------|-------|-------|
|            |         |         | K2   |       |       |
| opt-       | PG70-16 | 3.85    | 3.03 | 4.35  |
|            | PG64-16 | 3.86    | 3.35 | 3.23  |
| opt        | PG70-16 | 3.45    | 2.56 | 2.27  |
|            | PG64-16 | 3.57    | 2.78 | 2.44  |
| opt+       | PG70-16 | 2.86    | 2.38 | 2.22  |
|            | PG64-16 | 3.45    | 2.56 | 2.34  |
5. Conclusions

- The average resilient modulus (Mr), which belongs to a temperature of 5°C was 328036 psi revealing an approximate loss of 88% of its strength in terms of resilient modulus when there is an increase in temperature to 60°C. If linear regression is assumed between the resilient modulus value and temperature, the constant of proportionality will be approximately -5100 psi per 1-degree centigrade.

- The permanent deformation test result supports the fact of “the higher the pavement temperature, the rapid increase in the rate of plastic deformation”. The slope value that gives the accumulation rate of permanent deformation for 60°C is around three folds higher than that at 5°C. For the intercept, the corresponding value is approximately 268 percent.

- There is a distinctive difference in fatigue life due to the temperature change. The fatigue life decreases 32% with the rise in temperature from 5°C to 25°C.
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