Design and Laboratory Investigation of the Stone Mastic Asphalt Performance in Iran

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Abstract

Asphalt concrete is predominantly used as a construction material for flexible pavements. Besides all of the advantages of flexible pavements their resistant to heavy loading especially in regions with hot weather conditions is a matter of concern. In other words, low resistance to permanent deformation or so-called rutting would be the most important distress in such cases. Following to the recent nationwide survey that has been carried out by Iranian Ministry of Roads and Transportation, rutting was indicated as the major distress for Iran's highway system. The present study investigates design and laboratory evaluation of Stone Matrix/Mastic Asphalt (SMA) as an advantageous alternative mixture for construction of rutting resistant pavements. This type of mixture has been successfully used in many countries, e.g. European countries and United States of America. However, design of an appropriate SMA mixture based on the domestic technology and materials is still a challenge for highway administrations in different countries. Hence, this paper describes the main findings of the recent research activities in Iran to design four different SMA mixtures prepared with different nominal maximum aggregate sizes (NMAS) of 19 mm, 12.5 mm, 9.5 mm, and 4.75 mm. Uniaxial dynamic creep testing was carried out on different SMAs to evaluate the effect of NMAS and other factors on permanent deformation of the mixtures. Creep models were also extracted based on the results of the dynamic creep tests.

Keywords: *Flexible Pavements, Stone Mastic Asphalt, Permanent Deformation*.

Et Design Laboratory Investigation de la Performance d'asphalte coulé Stone (SMA) en Iran

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Résumé

Béton bitumineux est principalement utilise comme matériau de construction pour les chaussees flexibles .Les chaussees flexible sont tres resistantes sous les charges Lourdes en region chaude .En plus,elles resistent bien a deformation permanente causee par la charge sous la haute temperature.Les resultats recents des etudes nationales montrent que la deformation permanente causee par la charge sous la chaleur est la cause majeure des defections des autoroutes irraniennes .Dans l'etude presente nous avons montre que mastic Asphalt resiste bien au deformation permanente sous la charge dans la haute temperature. Ce mélange d'Asphalt a ete utilise avec succes en Europe et Etat-Unis .Mais trouve un mélange de mastic d'Asphalt base sur materieux et la technology domestique est une performance a faire.Donc dans ce document ,nous allons etudier l'effet de la dimension de gravier, 19mm,12.5mm 9.5mm et 4.5mm en glissement des Asphaltes par l'appareil .L'appareil de mesure des glissements uniaxials dynamics etait utilise pour mesurer les deformations permanents des echantions .A la fin un model mathematique pour des glissements etait dedui.

Mots-clés français*: chaussées souples,SMA, la déformation permanente.*

1. INTRODUCTION

With regard to the vast amount of budgets spend annually to maintain a good quality highway system, highway administrations are intended to employ more durable pavements on their infrastructural network. On the other hand, in most of the countries around the world the highway network is mainly constructed by flexible pavements rather than rigid pavements. For instance, in U.S over 90 percent of the paved roads are comprised of flexible pavements [1-3]. A survey shows that in Iran, similar to many countries, more than 94 percent of the highways are paved by flexible surfaces [4]. Asphalt concrete is predominantly used as a construction material for flexible pavements. Besides all of the advantages of flexible pavements their resistant to heavy loading especially in regions with hot weather conditions is a matter of concern. In other words, low resistance to permanent deformation or so-called rutting would be the most important distress in such cases. Following to the recent nationwide survey that has been carried out by Iranian Ministry of Roads and Transportation, rutting was indicated as the major problem with Iran's highway system [5]. Hence, design of mixtures that are resistant to permanent deformation was set as the main objective of the research presented herein. Two approaches were determined to be an aid to prevent the rutting distress in flexible pavements in Iran: first, to use the Performance Grading, PG, systems for classification of the bitumen produced in the country and employing the appropriate bitumen grade for severe loading and climatic conditions; secondly, to design resistant mixtures to deformation. The first solution is related to petroleum industries research and depends on the availability of the appropriate infrastructures. However funded researches are ongoing to prepare the PG map for Iran [6], this approach could not be useful for the vast amount of the present highway construction activities. The second approach, which is the focus of this study, would be of great advantage both before and after completion of the PG investigations. Hence, one of the best options would be Stone Matrix/Mastic Asphalt (SMA). This type of mixture has been successfully used in many countries. SMA was first developed in Germany in 60's and then was introduced to United States at 90's [7]. Since then, many research activities have been carried out to investigate the performance of such mixtures both in the laboratory and field scales. Ishai in et. Al. [8] and Mogawer et. al. [9] investigated the effects of fillers on SMA mixtures; several reports and articles were published by Brown and his colleagues in the case of SMA experience over the United States of America [10-12]; Recommendation and guideline for design of SMA mixtures by NCAT [13]; Investigation of appropriate laboratory test methods for SMA [11, 14]; Effect of aggregate size and shape on SMA performance; Determination of optimum compaction levels [14]; application of numerical modeling and image processing in the case of SMA mixtures [15, 16] are some highlighted researches. However, for the purpose of SMA mixture design there are still some challenges namely, design of mixtures by means of the domestic materials and engineering experience available in each country, investigation of the potential of different test methods to evaluate the performance of the mixtures, and development of new models and virtual tests [16- 18]. For instance, some researches indicated that the common repeated loading axial testing of bituminous mixtures may have some inadequacies in evaluation of mixtures by different aggregate skeleton, e.g. HMA versus SMA [19]. However, these tests would be employed to extract creep models which can be used in Superpave design. There are several models for this purpose while this study employs the most common method known as linear logarithm-logarithm empirical model [20].

Based on the facts mentioned above, the present paper discusses the design and Laboratory performance evaluation of four different stone mastic asphalt mixtures prepared by means of available materials and technology in Iran. The paper also elaborates upon its findings and the results to conclude new facts in the field of pavement materials.

2. EXPERIMENTAL

2.1. Material selection

2.1.1. Aggregate and filler

As indicated by many references the material used in construction of stone mastic asphalt should meet higher requirement than those employed for ordinary dense grade asphalt concrete mixtures. For instance, following to the expected function of an SMA mixture to carry the imposed loads by means of the stone-on-stone contact mechanism the coarse aggregate must be sufficiently strong. Low Los Angeles abrasion factor, high crushed aggregate faces, etc. must be met. Hence, the first step for design of SMA mixtures is to select appropriate materials. A primarily survey was carried out on different aggregate sources available in Tehran and its neighborhood provinces to find the aggregate source that best fits the minimum requirements presented in guidelines. In this study the NCHRP-9-8 [13] project was used in parallel to Iranian guideline for design of SMA mixtures, Issue 206 [21], as the reference material in design of SMA mixtures. Among all of the sources the Macadam-Sharq source located over Eastern borders of Tehran was determined as the most suitable aggregate source to be used in design and construction process. The associated aggregate properties are shown in Table 1.

Table1- *Quality tests results for aggregate used in this study*

Test Results			Test Method		Specification
Filler	Fine Agg.	Coarse Agg.			
-	76.0	-		(AASHTO-T176)	Sand Equivalent
-	-	B	Gradation type	(AASHTO-T96)	Los Angeles Abrasion Value
-	-	24	Percent loss		
2	N.P	-	Plastic index (PI)	(AASHTO-T89,90)	Atterberg limits
20	-	-	Plastic Limit (PL)		
22	N.A.	-	Liquid Limit (LL)		
-	-	100	One or more	(ASTM-D5821)	Percent Fractured faces
-	-	97	two or more		
-	-	16	Elongated particles	(BS-812)	Flat and Elongated Particles
-	3.1	-	Fine aggregate	(AASHTO-T104)	Sodium sulphate soundness
-	-	0.8	Coarse aggregate		

Four types of gradations with NMASs of 19 mm, 12.5 mm, 9.5 mm, and 4.75 mm were selected to prepare the SMA mixtures. Three trial gradations were prepared for each NMAS. The three trial blends were along the coarse and fine limits of the gradation band along with one gradation falling in the middle. Figure 1 shows the selected gradations used in this study in 0.45 power scale. More details about the process of choosing one among each trial gradation would be presented in the following section about SMA mix design.

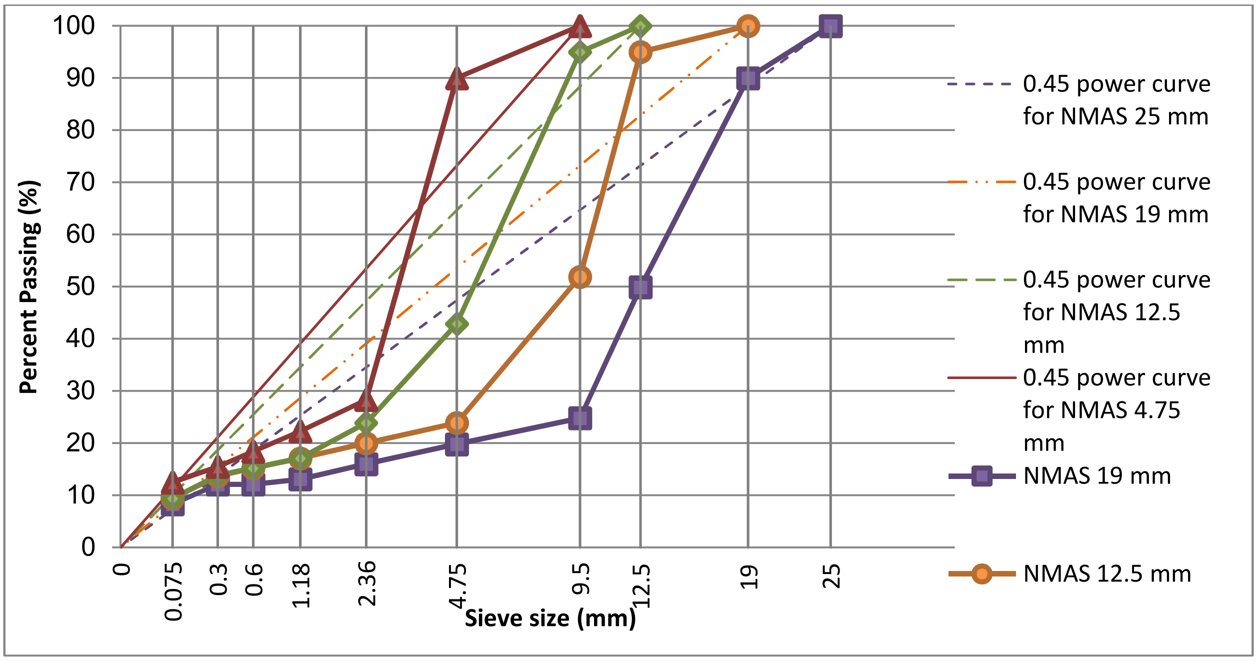

Figure 1- *Gradation of SMA mixtures with four different NMASs, 0.45 power scale.*

2.1.2. Bitumen used and Mastic properties

Aforementioned in introduction section, penetration grading system is common at the present time to categorize the bitumen produced in Iran's refineries. The unmodified bitumen selected to be used in this study was 60-70 penetration grade bitumen obtained from Pasargad Oil Refinery Co. located in Tehran, Iran. It was selected as it is in most common use in Iran. The conventional binder tests were conducted following to AASHTO standard test methods, as shown in Table 2. All the properties were inside the limits defined for 60-70 bitumen by the standards.

Table 2- *Traditional Asphalt binder test results.*

	Specifications for 60/70 penetration grade		Results	Test Method
Test	Lower Limit	Upper Limit		AASHTO
Specific Gravity at 25° C	-	-	1.017	T228
Penetration (0.1 mm)	70	60	61	T49
Softening Point (° C)	56	49	50	T53
Ductility (cm)	-	100	Over 100	T51
Solubility in Trichloroethylene (%)	-	99	99.2	T44
Flash Point (° C)	-	232	315	T48
Kinematic Viscosity at 120 ° C (cS)	-	-	870	T201
Kinematic Viscosity at 135 ° C (centistokes)	-	-	386	T201
Kinematic Viscosity at 160 ° C (cS)	-	-	135	T201
Thin film Oven test	0.8	-	0.02	T179

Besides the net bitumen properties, the associated asphalt mastic characteristics play important role in performance of SMA mixtures. Asphalt mastics are defined as a dispersion of aggregate fines of different sizes within a medium of asphalt binder [23]. No unanimity among researchers has been reached over the size limits of the aggregates friction that would constitute the mastic inclusion. They are commonly being defined based on the research application [24]. In this study standard sieve no.30 was selected to be the divider of the gradation fraction to be considered as the aggregate phase. The aggregate fraction passing standard sieve no.30 was added to the binder by 30% in volume. The mix was blended in 163°C with a high shear mixer to provide a uniform distribution of aggregate inclusion through the matrix. The cellulose fiber was also added during the blending process by 0.3% of binder weight. The mixing temperature was kept controlled at a steady amount of 163 ±1 °C. As illustrated by NCAT report [13] it is believed that the cellulose fiber would not significantly affect the viscoelastic properties of asphalt binder. In other words, the purpose of its usage is to prevent draindown as the result of higher bitumen content in SMA rather than modification of asphalt rheology. In addition to the available data for bitumen used that was provided by Pasargad Oil Company, a series of supplementary tests including Dynamic Shear Rheometer (DSR) tests were conducted both on original bitumen and mastic samples. The test was carried out in accordance with ASTM D7175-05 [25]. The strain controlled mode was used through DSR tests. The results for the standard frequency of 10 rad/sec are presented in Table 3. However it is suggested not to use the test results in the temperature ranges lower than 40°C and higher than 70°C but this work also uses the results for 30°C as the properties was still linear elastic. It can be recognized from the results that the parameter $G^*/\sin\delta$ is greater than 5 for all the specimens that would satisfy the recommendation by the NCHRP-9-8. It should be also noted that the arbitrary non-standard temperatures used in supplementary tests were in the line with the plan for a numerical modeling purpose which is out of scope of the present paper. However, the results are useful for the purpose of comparing the rheological changes.

Table 3- *DSR summary results for Mastic at standard frequency of 10 rad/sec.*

	Mastic samples				Original Binder
Temperature	30	54	66	82	52
G* (kPa)	880.57	124.825	26.206	1.2972	7.329
(Degrees)δ	43.02	58.95	64.25	78.88	84.00
$G^*/Sin(\delta)$	1290.679	145.7015	29.0952	1.322	7.3694

2.2. Design of SMA mixes

For the purpose of designing bituminous mixtures in Iran, Marshal Mix design method is in common use. This section describes the implication of Marshal method to design SMA mixtures. After choosing the appropriate materials, e.g. aggregate, bitumen, additives, etc. as presented in previous section, and preparation of the trial blends for each NMAS, the next step is to opt among the blends. For this purpose, three trial gradations, which fall along the coarse and fine limits of the gradation band along with one gradation falling in the middle, were initially evaluated. These gradations were based on percent volumes passing the respective sieves; however then were converted to gradations based on mass for the mixture design process. For each of these gradations at least three Marshal specimens were prepared by means of their associated trial bitumen content. The trial bitumen content was selected based on the bulk specific gravities of each blend as shown in Table 4. The relation among the bulk specific gravity of the aggregate and the trial bitumen content is also shown at Table 5. The trail bitumen content of 6.6 % was chosen to be appropriate for all the gradations. The specimens were then prepared by 50 blows per face of a flat face Marshall Hammer.

Table 4- *G_{sb} values of the aggregate with NMAS 12.5 mm in this study.*

19 mm			12.5 mm			9.5 mm			4.75 mm		
Lower	Middle	Upper	Lower	Middle	Upper	Lower	Middle	Upper	Lower	Middle	Upper
2.525	2.523	2.520	2.518	2.516	2.514	2.510	2.508	2.507	2.515	2.515	2.514

Table 5- *Associated trial bitumen content for different G_{sb} values (After NCHRP 9-8).*

G_{sb}	2.40	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85	2.90	2.95	3.00
Minimum bitumen Content (%)	6.8	6.7	6.6	6.5	6.3	6.2	6.1	6.0	5.9	5.8	5.7	5.6	5.5

Then, the specimens have been evaluated to meet the three criteria, namely VCA_{mix}, VMA, and V_a. For best performance, the SMA mixture must have a coarse aggregate skeleton with stone-on-stone contact. An accepted method for checking the presence of such aggregate skeleton condition is to calculate the Voids in Coarse Aggregate fraction (VCA) both for dry rodded condition of the aggregate and for the compacted SMA specimen. In other words, the condition of stone-on-stone contact within a SMA mixture is defined as the point at which the voids in coarse aggregate (VCA) of the compacted mixture is less than the VCA of the coarse aggregate in the dry rodded test according to AASHTO T19 standard method. The VCA_{DRC} can be calculated using the Equation 1. The results are presented at Table 6.

$$VCA_{DRC} = \frac{G_{CA}\gamma_{\omega} - \gamma_{s}}{G_{CA}\gamma_{\omega}} \times 100 \quad [1]$$

$$VCA_{MIX} = 100 - (\frac{G_{mb}}{G_{CA}} \times P_{bp}) \quad [2]$$

$$P_{bp} = P_s \times PA_{bp} \quad [3]$$

Where: VCA_{DRC} is the voids in coarse aggregate in dry-rodded condition, VCA_{mix} is the VCA of the compacted mixture, P_{bp} is the percent of aggregate by weight of the mixture remaining on the breaking point sieve, P_s is the percent of aggregate in the mixture, and PA_{bp} is the percent of aggregate by the total weight of the aggregate remaining on the breaking point sieve. The summary of the results are presented in Table 6. On the basis of the results it was concluded that the optimum gradation blend for NMASs of 19 mm, 12.5 mm, 9.5 mm, and 4.75 mm are following the lower band, middle band, middle band, and the lower band, respectively.

Table 6- *Summary of SMA mix design data.*

NMSA	Gradation	AC (%)	GCA	G_{sb}	G_{mb}	G_{mm}	V_a	VMA	VCA_{drc}	VCA_{mix}	VCA ratio
19 mm	Lower	6.6	2.535	2.525	2.236	2.323	3.73	17.55	39.88	34.14	0.86
	Middle	6.6	2.535	2.523	2.269	2.326	2.45	16.29	40.4	36.51	0.91
	Upper	6.6	2.535	2.520	2.277	2.307	1.28	15.86	39.92	39.53	0.99
12.5 mm	Lower	6.6	2.521	2.518	2.149	2.340	8.17	20.53	40.78	36.42	0.89
	Middle	6.6	2.521	2.516	2.212	2.320	4.64	18.13	40.14	37.82	0.94
	Upper	6.6	2.521	2.514	2.238	2.311	3.15	17.11	40.74	40.32	0.99
9.5 mm	Lower	6.6	2.513	2.510	2.156	2.318	6.99	20.03	40.03	36.02	0.90
	Middle	6.6	2.513	2.508	2.202	2.316	4.92	18.26	39.11	37.92	0.97
	Upper	6.6	2.513	2.507	2.215	2.299	3.65	17.75	39.87	40.84	1.02
4.75 mm	Lower	6.6	2.506	2.515	2.160	2.309	6.47	20.04	41.34	37.65	0.91
	Middle	6.6	2.506	2.515	2.180	2.318	5.95	19.3	40.82	42.74	1.05
	Upper	6.6	2.506	2.514	2.144	2.267	5.39	20.59	41.18	49.17	1.19

The optimum gradations were then used to determine the optimum bitumen content. In addition to the trial bitumen content, for each gradation two other compacted specimens were prepared by other two different bitumen contents. The increments of 0.4 % by weight were used as recommended by the guidelines. In addition to these compacted samples one loose specimen was also prepared per each of the bitumen contents for the purpose of maximum density and air voids calculations. The optimum bitumen content for each of the gradations were found to be 6.5%, 6.8%, 6.9%, and 7.7% for the NMASs of 19 mm, 12.5 mm, 9.5 mm, and 4.75 mm, respectively. As expected, it was recognized that the optimum bitumen content was inversely related to the nominal maximum aggregate sizes of the gradations. These relatively high bitumen contents would make the mixture susceptible for some distresses like draindown which is discussed in the following section.

2.3. Draindown Sensitivity Test

Homogeneity is a key aspect in quality of the constructed SMA pavements. As a result of the significantly higher bitumen content in SMA mixtures it is strongly possible that the mastic phase segregates from the mixture and flow at the bottom of the finishers. This would lead to an inhomogeneous mixture with a weak performance. The additives, e.g. cellulous fibers employed in this study, are commonly used to avoid such an undesirable consequence of the excess free bitumen. So, the last step was to evaluate the sensitivity of the mixtures to draindown. This test was carried out in two different temperatures, anticipated production temperature and 15°C higher than the production temperature, following to the AASHTO T305 test method [26]. The first temperature was determined to be 152°C following to the AASHTO T245. Consequently, the other temperature would be

167°C, which is considered to be 15°C above the plant temperature. The test was carried out on two uncompacted specimens for each temperature. Table 7 shows the results of the draindown sensitivity test. It can be recognized from the table that none of the results exceed the maximum controlling limit of 0.30.

Table 7- *Draindown sensitivity test results.*

Test Temperature (°C)	19 mm	12.5 mm	9.5 mm	4.75 mm
152	0.039	0.052	0.026	0.140
167	0.062	0.100	0.052	0.21

2.4. Sample Preparation

All of the four different SMA mixtures were then studied about the permanent deformation characteristics at the laboratory level. Hence, three replicates of laboratory specimens were prepared for each mixture in accordance to the requirements by the BS-DD226 for dynamic creep tests. This standard method uses slightly different conditions to evaluate the permanent deformation of the bituminous mixtures under repeated loading. The specimens were prepared in cylindrical metal molds compacted by 50 blows per face of flat-faced Marshall hammer for SMA mixtures. The top and bottom circular faces of the specimens were then smoothed mechanically by means of a rotary cutter to avoid any undesirable effects on records and the associated data scattering. The specimens were then allowed to rest for 48 hours prior to testing. Figure 2 shows schematics of prepared specimens in the laboratory.

Figure 2- *View of different laboratory prepared specimens (SMA & HMA).*

3. RESULTS OF DYNAMIC CREEP TEST

3.1. Creep test Procedure

In order to evaluate the permanent deformation characteristics of the designed mixtures in this study the dynamic creep test was employed. This test was carried out in accordance with the British standards method as mentioned in BS-DD226. The aforementioned specification test temperature is at 30°C; however higher temperature would be used to further investigate the properties of the mixtures. In addition to the standard testing temperature, 30°C, a higher temperature of 45°C was also used to investigate the deformation properties. Dynamic loading was applied in trapezoidal shape to 100 mm diameter cylindrical specimen. Through the test axial stress pulses of 1 second duration and magnitude of 100 KPa separated by 1 second rest periods was applied to the specimens. The test duration was 1800 load pulses or 3600 seconds and it was carried out at a constant temperature in an isolated test chamber. The test was carried out by means of the UTM-14P equipments available at Technical Soil Mechanics Laboratory of the Ministry of Roads and Transportation of Iran. A view of the testing equipments is shown in Figure 3. The detailed discussion on the results of creep tests are presented as the following sections.

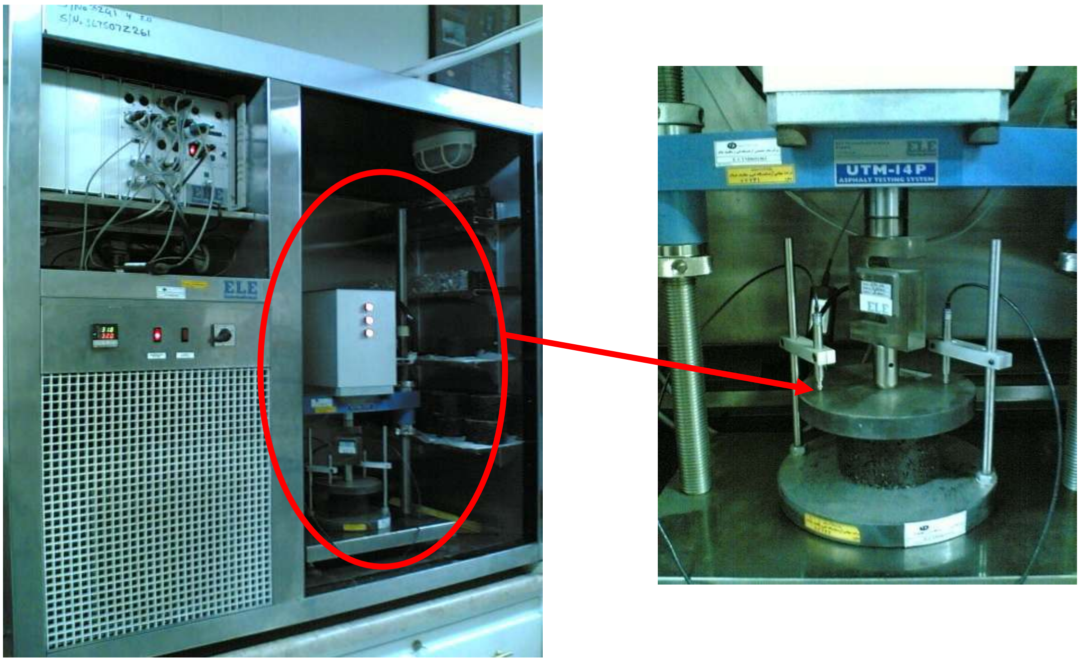

Figure 3- *View of the Unconfined Dynamic Creep testing apparatus, UTM-14P equipments.*

3.2. Changes in deformation relative to Aggregate Skeleton

A key parameter in deformation characteristics of bituminous mixtures is the way that they bear applied loads. In SMA cases, this load bearing mechanism is assumed to be carried by means of stone-to-stone contact through the aggregate skeleton. This mechanism is significantly different in case of HMA mixtures. The results of creep tests presented in

Figure 4 indicated that the accumulated permanent deformations of different SMA mixtures are in reverse order with the Nominal Maximum Aggregate Size (NMAS) of the gradation used for the mixture. Based on the results of this study, it has been recognized that the smaller NMAS would yield higher resistance to permanent deformation through the creep tests at both 30 °C and 45 °C. It should be noted that the trend of changes is uniform as the permanent deformation decreases by growing of the NMAS. Therefore, the SMA mixtures prepared by NMAS of 4.75 mm was determined as the most resistant mix followed by those of 9.5 mm, 12.5 mm, and 19 mm, respectively. Literature also confirms that there should be such a reasonable trend in changes of creep performance versus changes in NMAS. However, some researches indicate that based on the aggregate production method, the source of the aggregate and its surface texture quality it would be possible that an intermediate NMAS provide the best creep performance. Such cases were explained to be as a result of an especial internal structure situation by which the maximum aggregate interlocking would be obtained. Although based on the domestic materials and aggregate sources employed in this study such an extra interlocking was not observed, the possibility still exists for other aggregate sources. Altogether, the overall trend of changes can be expected in accordance with the observations in this study. Investigation on the average accumulated permanent strains shows that there would be a linear trend of changes between NMAS and the strains. Based on the results it could be claimed that in 30 °C the strain changes by a rate of about 6.6% while the associated rate in 45 °C was found to be 10 %. The find outs confirms the expected change as a result of the temperature sensitivity of bituminous materials.

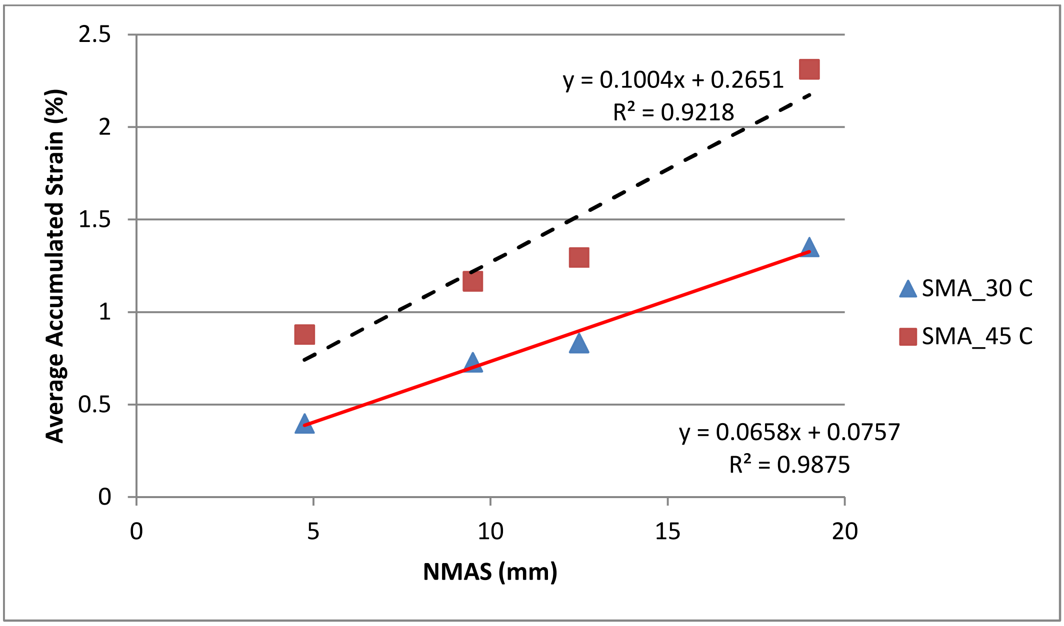

Figure 4- *Rate of the changes of strain vs. NMAS of SMA mixtures.*

3.3. Creep Performance by Changes in Temperature

Following to the basic definition of the viscose materials, it is clear that the deformation related performance of bituminous mixtures is strongly dependent on the changes in temperature. This section discusses the impact of change in the repeated loading creep testing temperature from 30°C, which is the default temperature in DD 226 standard, to 45 °C. Aforementioned in the previous section the rate of changes in 45 °C is greater than that of 30 °C. Hence, sensitivity to changes in aggregate skeleton would also increase in higher temperatures. On the other hand for each of the NMASs there could be seen a considerable amount of changes in accumulated strain. As illustrated by Figure 5, the accumulated strain in different mixtures increases from a minimum of 55 % up to a maximum of 120 % by 15 °C change in test temperature. Subtracting the accumulated strains in 45 °C from the associated values in 30 °C would provide an estimate of the excess strain as the result of temperature change. Dividing these excess strain values by the strain values in 30 °C the increase strain would be calculated as percentages of the lower temperature results as the base value. These percentages are shown as labels on the top of each column bars in Figure 5. It could be observed that in general the greater NMASs would result in lower sensitivity to temperature changes. The 19 mm mixture is the only exception; however it is also less sensitive than the 4.75 mm mixture. Based on such a trend it could be recognized that at the higher temperatures the accumulated strain of the smaller NMASs would reach close to coarser gradations. However, it could be concluded that the finer gradations would lead to SMA mixtures that provide a better creep performance in a wider temperature changes range, from intermediate to high temperatures. All of the parameters discussed hereby play considerable roles to make the administrative decision about the best option for construction. For instance the finer gradation is accompanied by using a higher bitumen content which means higher initial cost of construction.

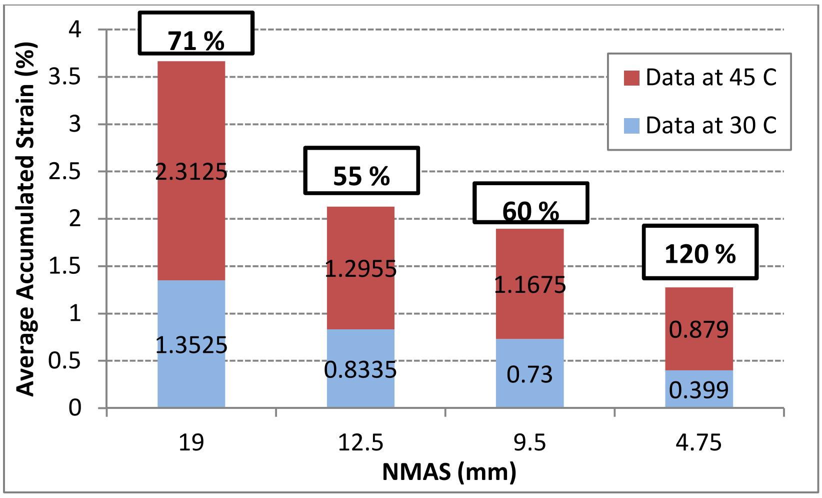

Figure 5- *Increased accumulated strain from 30° C to 45° C and the associated ratios.*

3.4. Creep models

This section describes the analysis carried out on the data obtained from dynamic creep tests. Although there have been numerous models proposed to describe the relationship between permanent strain (ε_p) and number of load repetitions (N); the empirical linear log ε_p - log N relationship is the most popular model used in the literature. This model has also been used in the SHRP (Superpave) developed mix design and analysis procedure under level 2 and 3 conditions [3]. The relationship can be mathematically expressed by the Equation 4.

$$\varepsilon_p = aN^b \quad [4]$$

Where: ε_p is the permanent strain, N is the number of load repetitions, “a” and “b” are material properties related parameters which are intercept and slope from regression, respectively.

For the purpose of extracting the empirical models for any of the mixture types, the creep results were prepared in fully logarithmic scale. Some regression based analysis was conducted on the results. The results from regression are presented in Table 8.

Table 8- *Regression based creep parameters “a” and “b”.*

Mixture Type		30 °C				45 °C			
		NMAS 19	NMAS 12.5	NMAS 9.5	NMAS 4.75	NMAS 19	NMAS 12.5	NMAS 9.5	NMAS 4.75
Creep Parameters	a	0.350	0.142	0.170	0.124	0.106	0.206	0.155	0.182
	b	0.128	0.171	0.144	0.110	0.330	0.203	0.219	0.143

From Equation 4 it could be recognized that the permanent strain would increase by any growth in both “a” and “b” parameters. However, for the cases that these two regression based parameters change in reverse order no certain judgment could be made. In such cases the resultant change in strain is dependent on the effect of the two contradictory changes of the “a” and “b” parameters. Appropriate charts were developed to distinguish the changes in creep model parameters versus NMAS for both 30 °C and 45 °C temperature. It could be concluded that in general the smaller NMAS results in smaller “b” parameter values. Elaborating upon the regression shows that the “a” parameter follows a different trend. It was observed that in 30 °C the average changes in “a” value follows the same pattern as “b” parameter, while for 45 °C the trend of changes turned into reverse order. In other words, for the greater NMASs the “a” values degrades in 45 °C. By comparing the trend of changes of the two parameters under the different testing temperature investigated in this study it was concluded that both the parameters play in the same direction in 30 °C. On the other hand, it was recognized that the 45 °C testing results are different in a way that the changes of parameters “a” and “b” tend to neutralize each other. More investigation showed that under the testing conditions of this research, e.g. stress level, temperatures, loading characteristics, the governing parameter was “b”.

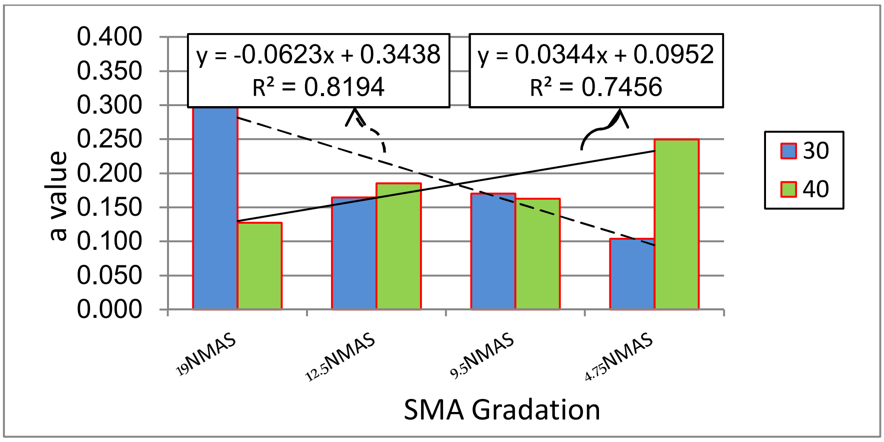

Figure 6- *Changes in “a” parameter versus NMAS.*

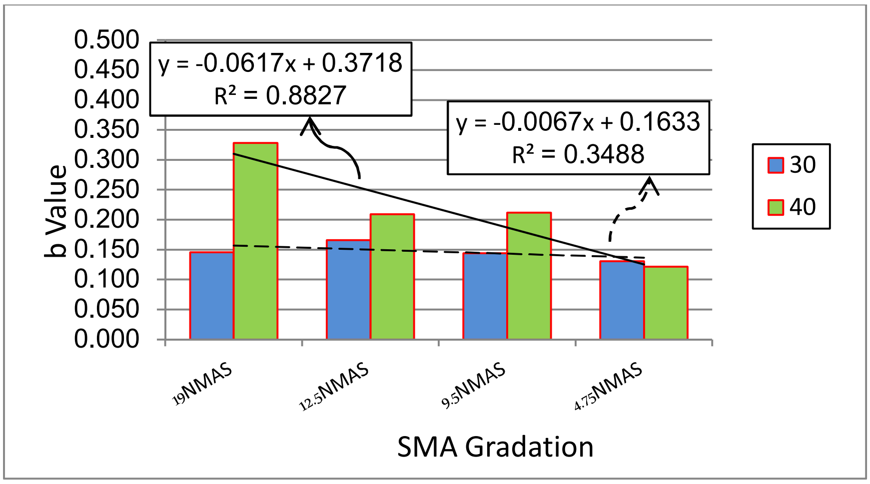

Figure 7- *Changes in “b” parameter versus NMAS.*

4. CONCLUSION

With regard to the importance of durable transportation infrastructures this paper focused on design and laboratory evaluation of Stone Mastic Asphalt mixtures by means of domestic materials and technology in Iran. It can also provide some indicates of the deformation performance criteria for gradations with the same source and different NMAS. Based on the results of temperature sensitivity, overall accumulated strain, and bitumen content, and consequently budget restraints it would provide the tradeoff between different SMA mixtures and a comparison to ordinary HMA mixtures. The appropriate aggregate source which was used as the prerequisite of the present study would be applicable for the Capital Province of Tehran and some of its adjacent neighborhoods. The result showed that Marshall method could be successfully employed to design the SMA mixtures. It was concluded that there is a direct relation between the NMAS and accumulated strain of SMA mixtures. In other words, it was found that lower NMAS resulted in higher resistant to permanent deformation. It was also recognized that in general, the finer gradation in SMA would have higher sensitivity to temperature changes. For all the SMA mixtures the associated linear logarithmic empirical creep models were also extracted. The results indicate that parameter “b” is reversely related to NMAS, while the parameter “a” seems to have different trend by changing the temperature. However for the mixtures investigated in this study it was determined that the governing parameter in the empirical model is “b”. It was also confirmed that the repeated loading axial test in accordance with British Standards is able to discriminate among different SMA mixtures. The arbitrary chosen temperature of 45 °C would provide similar creep performance to the standard test temperature of 30 °C in British standards.

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