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Article

Permanent Deformation Behavior under Repeated Load of Recycled Material Stabilized with Bitumen Emulsion

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Abstract. Cold in-place recycling (CIR) with bitumen emulsion is a method for enhancing resistance to permanent deformation damages of flexible pavement. The objective of the study was to examine the permanent deformation behavior of a bitumen-stabilized material (BSM) subjected to repeated loads. To achieve this, the aggregate was derived from reclaimed asphalt pavement and reclaimed crushed rock in the ratio of 25:75 mixed with 2% or 3% bitumen in emulsion and 1% Portland cement (of the total aggregate weight). After that, the specimens were subjected to repeated loads following the EN13286-7 method. The results were as follows. At the compressive deviatoric stress level of 550 kPa, All of BSM specimens with 2% and 3% bitumen exhibited the plastic shakedown equilibrium pattern of permanent deformation after 1,000,000 cycles of repeated loads and their permanent strain were less than 0.6%. The results also indicated that once the permanent strain rate decreased to 0.004 microstrain/cycle or lower, the BSM became stable in the plastic shakedown state. The findings in this study may serve as guidelines for designing pavement rehabilitation with BSM to prevent permanent deformation caused by traffic loads.

Keywords: Permanent deformation, bitumen emulsion, stabilized material, repeated load, shakedown.

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1. Introduction

In Thailand, about 88% of highways were constructed using asphalt material [1]. And cold in-place recycling (CIR) had become a familiar method for rehabilitating asphalt pavement [2]. Generally, the CIR method predominantly employed 5-10% cement as the stabilizing agent [3]. Although this was able to enhance material cohesion and increase the modulus to a high level of 10,000 MPa [4], the long-term use of cement-stabilized material suffered from reflective cracks in the asphalt layer. This was possibly from shrinkage cracks and excessive vertical stresses caused by the higher modulus in the cement stabilized material [5]. As the cracks increased intensely in the base layer, reflective cracks to pavement surface occurred.

In addressing such problems, one common solution is to mix bitumen, the main stabilizing agent, with a slight portion of cement. Dissimilar to the traditional asphalt concrete material, this alternative material relies on a small amount of bitumen that is only sufficient to enhance the cohesion between aggregate particles without coating their whole surfaces. As a consequence, not only does the material achieve a high yet non-excessive modulus level [6], but it is also effective in resisting humidity [7, 8].

The pavement stabilization approach mentioned above involves enhancing reclaimed asphalt pavement and existing aggregate by mixing them with a bitumen-stabilizing agent to create a new base layer material [5]. In applying the technique, it is vital to be aware of the material resistance to the repeated loads resulting from weight and traffic volumes [9, 10]. In Thailand, BSM has not been employed in pavement rehabilitation due to lack of local BSM specification. Thus, the objective of this study is to earn a better understanding of BSM behavior in resisting permanent deformation and provide a benchmark to the pavement application.

2. Literature Review

2.1. BSM

BSM contains 1.5-3.5% bitumen – a low amount that does not coat the whole aggregate surface. For this reason, it is a non-continuously bound material whose force transfer behavior resembles that of an unbound material, such as crushed rock or gravel, often used to lay the base and subbase layers. One major drawback of this non-continuously bound material is its vulnerability to permanent deformation [4] as evidenced by pavement subsidence and rutting. Another important concern is its shear resistance behavior enabling it to withstand repeated loads [11, 12, 13, 14].

Jitareekul analyzed a BSM applying the reclaimed asphalt pavement (RAP) to reclaimed crushed rock (RCR) ratios of 75:25, 50:50, and 0:100 [15]. It was found that the presence of RAP reduced resistance to permanent deformation. In addition, when the levels of repeated loads reached 400 kPa, the material containing 50% RAP exhibited similar permanent deformation resistance to that comprising only the RCR.

In their research, Sufian et al. tested the effects of a cement-bitumen mixture on material performance in three aspects, namely unconfined compressive strength, indirect tensile strength, and resilient modulus. The team varied the RAP to RCR ratios from 100:0 to 75:25, 50:50, 25:75, and 0:100 [6], discovering that the mixture was able to enhance all three aspects of material performance across all the aggregate mixture ratios. The findings resonated with those of an earlier study reporting that adding 1% cement in conjunction with bitumen resulted in better performance of BSM [7].

Flores et al. [16] studied the design methodology of in situ cold recycled mixtures with bitumen emulsion and RAP, it was found that adding bitumen emulsion to improve RAP material improved fatigue life. Moreover, increasing the cement content to improve RAP material increase stiffness modulus and resistance of permanent deformation.

Hornych et al. [17] tested repeated load triaxial on gravel mixed with bitumen emulsion. Adding bitumen emulsion in gravel decreased internal friction particle. Therefore, resistance permanent deformation of gravel-emulsion is decreased. Furthermore, increasing frequency of loading increases resilient modulus.

In repeated load triaxial test with cold-recycling bituminous mixtures, Santagata et al. [18] found resilient modulus of unbound granular material mixed with bitumen emulsion and cement increasing when temperature decreased. Additionally, the curing time improved resilient modulus of mixtures.

2.2. Permanent Deformation Behavior

The shakedown theory [19] has been widely adopted in research involving resistance to pavement permanent deformation from repeated loads [20-23]. Werkmeister has described critical repeated loads as the force that a material needs to be able to withstand in order to retain its shape during its life-time [19]. If the critical repeated loads are exceeded, permanent deformation arises. Known as the shakedown limit, the ability of a material to resist repeated loads can be divided into four patterns of behavior, as displayed in Fig. 1.

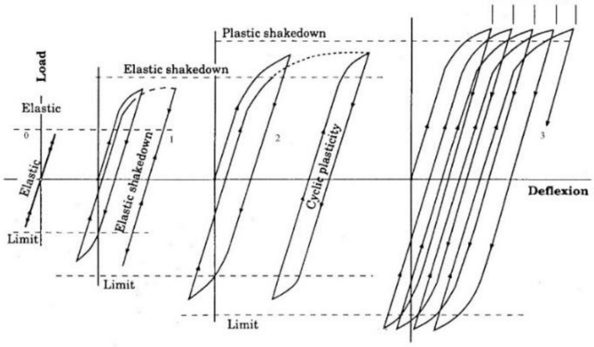

Fig. 1. Patterns of material behavior under repeated loads [19].

The purely elastic pattern (0) is when each particle of a material receives insufficient repeated loads to drive it to a yield point, and consequently, the material can still elongate or contract fully.

The elastic shakedown pattern (1) occurs when each particle of a material receives insufficient or limited repeated loads to drive it to the plastic shakedown limit. As the material does not fully enter the plastic shakedown state, it eventually returns to the purely elastic state.

The plastic shakedown pattern (2) takes place when the repeated loads to which a material is subjected are insufficient to trigger the ratchetting permanent deformation. In other words, the material is slightly affected by the accumulated permanent deformation caused during each cycle of repeated loads, but it can still retain its shape in the long run.

The incremental collapse pattern (3) arises when the repeated loads exerted on a material are high enough to drive it to or beyond a yield point. Since the accumulated permanent deformation proceeds quickly and without restriction, the material collapses momentarily.

Based on the above theoretical concept, it is possible to further determine the maximum repeated loads that a material can withstand without reaching its permanent deformation state. For flexible pavement, when the maximum repeated loads is exceeded, the accumulated permanent deformation or rutting occurs as a result of the mechanism associated with the incremental collapse pattern. In contrast, if the maximum repeated loads are not exceeded, the material will initially compress before undergoing a transition to the state associated with the elastic or the plastic shakedown pattern. After this state, the material will exhibit resilient strain behavior [20].

However, the phenomena presented in Fig. 1 characterizes ideal theoretical behavior that may not express perfectly in laboratory or real-world settings. In the study, the behavior of granodiorite crushed rock involving 80,000 to 2,000,000 cycles of repeated loads was analyzed. It was found that at lower degrees of repeated loads, the specimens exhibited increasing accumulated permanent deformation but did not collapse. In contrast, at higher degrees of repeated loads, they underwent accumulated permanent deformation to the point of collapse. The patterns of permanent deformation behavior found in the research fell into the following three types [19].

In the plastic shakedown pattern, the material under investigation initially underwent accumulated permanent deformation, exhibiting resilient strain behavior. Then it entered a state of balance, characterized by the straight line in Fig. 2. To put it simply, the material exhibited slight permanent deformation arising from each cycle of repeated loads.

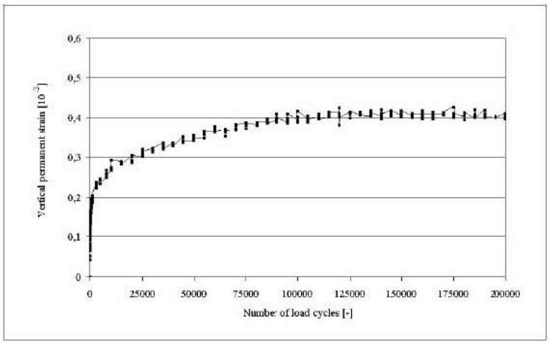

Fig. 2. Relationships between the number of repeated load cycles and permanent vertical strain under the plastic shakedown pattern [19].

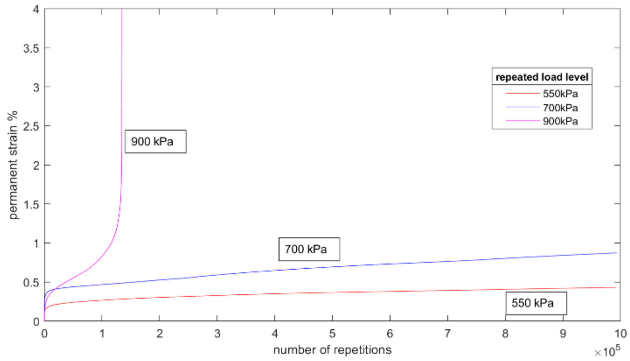

Fig. 3. An example of permanent axial strain responses under different levels of repeated loads.

The plastic creep pattern involved similar accumulated permanent deformation to that in the plastic shakedown pattern with the difference being that the increasing degrees could cause the material to collapse. This was consistent with the finding of Kolisoja [24], which reported that the accumulated permanent deformation remained relatively constant during the first 100,000 cycles of repeated loads but began to exert sufficient force to bring about collapse after 100,000 cycles of repeated loads.

The influences of increasing repeated loads from low to high magnitudes on the permanent deformation development patterns of the specimens are illustrated in Fig. 3.

2.3. Repeated Load Tests in Laboratory Settings

Various studies have been conducted to evaluate the long-term performance of material in laboratory settings with many involving over 1,000,000 cycles of repeated loads. For instance, Huurman tested the effects of 1,000,000 repeated load cycles or 10% permanent axial strain on subbase sand materials with the height of 200 mm and the diameter of 101.6 mm. It was indicated that sands only showed a strong development of permanent strains if they were loaded by the stress very close to the cyclic failure stress [25]. Similarly, Jenkins assessed the performance of a foamed BSM with the height of 250

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mm and the diameter of 150 mm subjected to 1,000,000 cycles of repeated loads or 4% permanent axial strain [26], applying continuous 2 Hz haversine repeated loads following the EN13286-7 standard. The applied deviator stress (σd) were varied between 30% to 70% of deviatoric stress at failure under monotonic triaxial test (σd,f). The permanent axial strains of the foamed BSM exhibited one of the three patterns shakedown patterns depending on the amount of applied deviator stress. It was concluded that there was a critical boundary of applied deviator stress ratio (σd/σd,f) for excessive permanent deformation. Below this ratio, less than 2% axial strain was observed after one million load repetitions. For foamed BSM with cement less than 2%, the critical ratio was 52%. Saberian et al. [27] had characterized the permanent deformation patterns of recycled concrete aggregate (RCA) blended with crumb rubber (CR) based on the shakedown theory. They conducted the permanent deformation test using haversine shaped deviator load and static confining pressure on the specimens for 15,000 cycles. The same cyclic stress (i.e. deviator stress) of 108 kPa and confining pressure of 38.7 kPa were applied to all studied cases. These stresses were selected to represent the average stress levels in a base course layer in a typical pavement structure. It appeared that amongst the various blends of RCA and CR, RCA and RCA+0.5%CR were classified as plastic creep pattern and could be used for pavement base.

3. Objectives and Scope of the Study

This study aimed to examine the behavior of BSM subjected to repeated loads in laboratory settings simulating the conditions of a base layer during traffic loads from standard axles following the EN13286-7 procedure [28]. It is expected that the findings will create a better understanding of the application of BSM in order to reduce or prevent permanent deformation. Based on the previous research work [29] on designing the composition of this BSM, the material composition of this research was as follows:

- The aggregate comprised of RAP and RCR in the weight ratio of 25:75.
- The effects of the bitumen content were assessed by varying 2% and 3% (of the total aggregate weight.
- 1% cement (of the total aggregate weight) was added to all BSM specimens.

4. Research Procedures

4.1. Materials

The bitumen emulsion used in this study was CSS-1h. The aggregate used in this research comprised of RAP and RCR in the ratio of 25:75. This was obtained by milling the existing asphalt pavement using a 2200CR Wirtgen milling machine at the speed of 5 m/min. The aggregate derived varied in sizes with the largest size of 25.4 mm according to the Department of Highways (DoH) Class A DH-S 201/2544 Standard [30], as displayed in Table 1. The modified compaction test according to ASTM D1557 [31] was conducted on the BSM aggregate and obtained maximum dry density of 2.297 g/cm^3 at 8.5% optimum water content.

Table 1. Aggregate size distribution.

Sieve sizes	**% Passing by weight of aggregate**	**% Passing according to the DoH Class A DH-S 201/2544 Standard**
50 mm (2 in)	100	100
25 mm (1 in)	100	-
19 mm (¾ in)	84.13	-
12.5 mm (½ in)	63.89	-
9.5 mm (3/8 in)	53.19	30-65
4.75 mm (size 4)	35.81	25-55
2. mm (size 10)	15.25	15-40
0.425 mm (size 40)	5.04	8-20
0.075 mm (size 200)	1.96	2-8

Table 2. Results of the unconfined compressive strength tests on the BSM specimens.

BSM composition	Unconfined compressive strength (kPa)
2% emulsion + 1%cement	1,298
3% emulsion + 1% cement	1,367

4.2. Specimen Preparation

Following the European Standard EN13286-7 [28] and Mulusa's method [32], the specimen was prepared from a cylindrical mold with diameter of 100 mm and height of 200 mm to achieve the diameter-height ratio of 1:2. In preparing the material for a specimen, the aggregate, cement and emulsion were weighed and then mixed together. Then the material was split into five equal portions for compacting five layers in the mold. Each material layer was compacted using a GSH 11E BOSCH vibratory hammer loaded with 30 kg weight as shown in Fig. 4. Between the adjacent layers, the surface of the prior layer was scarified well to allow for a homogeneous bonding between the layers [19].

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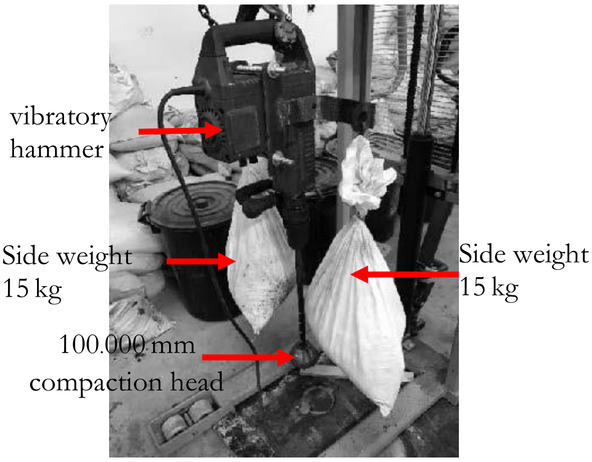

Fig. 4. Vibratory hammer used in compaction.

Fig. 5. Configuration of the experimental equipment.

After the above process was completed, the specimens had to reach 96.0-98.0% of the maximum dry density. Subsequently, the specimens were cured to ensure an equilibrium condition of moisture level following the procedures in the TG2 [7]. This step involved placing the specimens in a well-ventilated room with a temperature of approximately 30 °C for 26 hours, wrapping them using plastic with double the volume of the specimens, and curing them in an incubator set at 40 °C for 48 hours with the plastic wrap being changed every 24 hours. The specimens were left to cool down prior to the next experimental step with the plastic wrap remaining intact to prevent moisture loss.

4.3. Repeated Load Test

Lertpaichaiyon reported that the confining stress from traffic loads on a BSM ranged from 0 to 130 kPa and that the triaxial compressive strength of the BSM across such confining stress levels did not differ significantly from its unconfined compressive strength [29]. Based on the mentioned finding, the loading was applied in vertical direction without the confining pressure in this study. The results of unconfined compressive strength tests on the 2% and 3% BSM specimens with 1% cement are shown in Table2.

The specimen was setup in the servo hydraulic loading equipment shown in Fig. 5. The equipment comprised of a Servopulser, a load transducer, two displacement transducers (DT) and a data acquisition device connected to a computer for logging the electrical signals from the load and displacement transducers. The equipment set is illustrated in Fig. 5.

The equipment applied loads via the actuator attached to the bottom side of the specimen. On the upper side, the specimen was attached to the load transducer and to the loading frame. As such the vertical loads were applied from the bottom end of the specimen in the form of continuous haversine loads following the EN13286-7 [28]. Two displacement transducers were attached between the equipment frame and the specimen to monitor the specimen vertical deformation. During a test, the magnitude of the peak vertical compressive stress was kept constant through 1,000,000 repeated cycles or until the accumulated vertical permanent strain reached 10% according to the procedures undertaken in Huurman [25]. The repeated load tests were conducted at three levels of peak haversine stresses i.e. 550 kPa, 700 kPa, and 900 kPa. Three specimens were tested for repetition on each peak haversine stress level.

Table 3. Levels of repeated loads in the 2% and 3% bitumen emulsion tests.

Loading case	Repeated loads (deviator stress = σ_d) kPa
1	550
2	700
3	900

5. Results, Analysis and Discussion

Figure 6 displays a typical axial deformation response of a specimen recorded during the test. The displacement data was obtained at 50 samples per second. The deformation data was processed simultaneously to determine the lower peak of each cycle which was the accumulated permanent deformation of the specimen. The permanent axial strain and the permanent strain rate can be calculated using Eq. (1) and (2), respectively.

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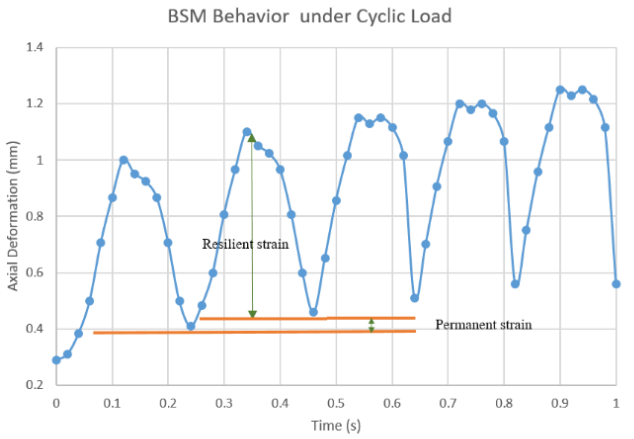

Fig. 6. Axial deformation of a BSM specimen recorded from a displacement transducer during the test.

$$\varepsilon_p = \frac{Permanent\ Deformation}{specimen\ height} \quad (1)$$

$$Strain\ rate_i = \frac{\varepsilon_{p,i} - \varepsilon_{p,i-1}}{n_{cycle}} \quad (2)$$

where $\varepsilon_{p,I}$ represents the permanent strain after the i cycle of repeated loads

$\varepsilon_{p,i-1}$ represents the permanent strain after the i-1 cycle of repeated loads

n_{cycle} represents the number of repeated load cycles from i-1 to i

Figures 7 and 9 present the relationships between the number of repeated load cycles and the permanent strain exhibited by the 3% BSM and the 2% BSM, respectively. Figs. 8 and 10 display the relationships between the number of repeated load cycles and the permanent strain rates exhibited by the 3% BSM and the 2% BSM, respectively. Tables 4 and 5 show the minimum strain rates, the number of repeated load cycles at the minimum strain rates, and the number of repeated load cycles at the end of the test, at different deviator stress levels for the 3% BSM and the 2% BSM, respectively.

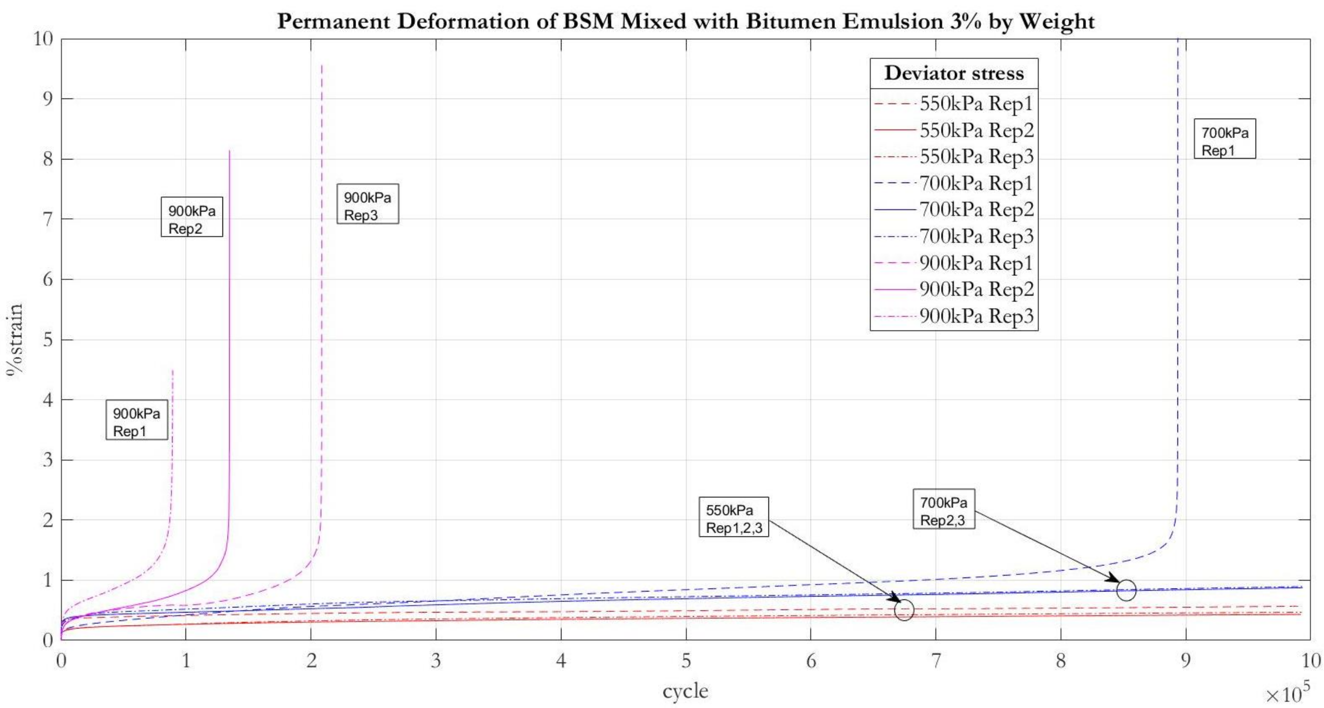

Fig. 7. Relationships between the number of repeated load cycles and the accumulated permanent strain of the 3% BSM specimens.

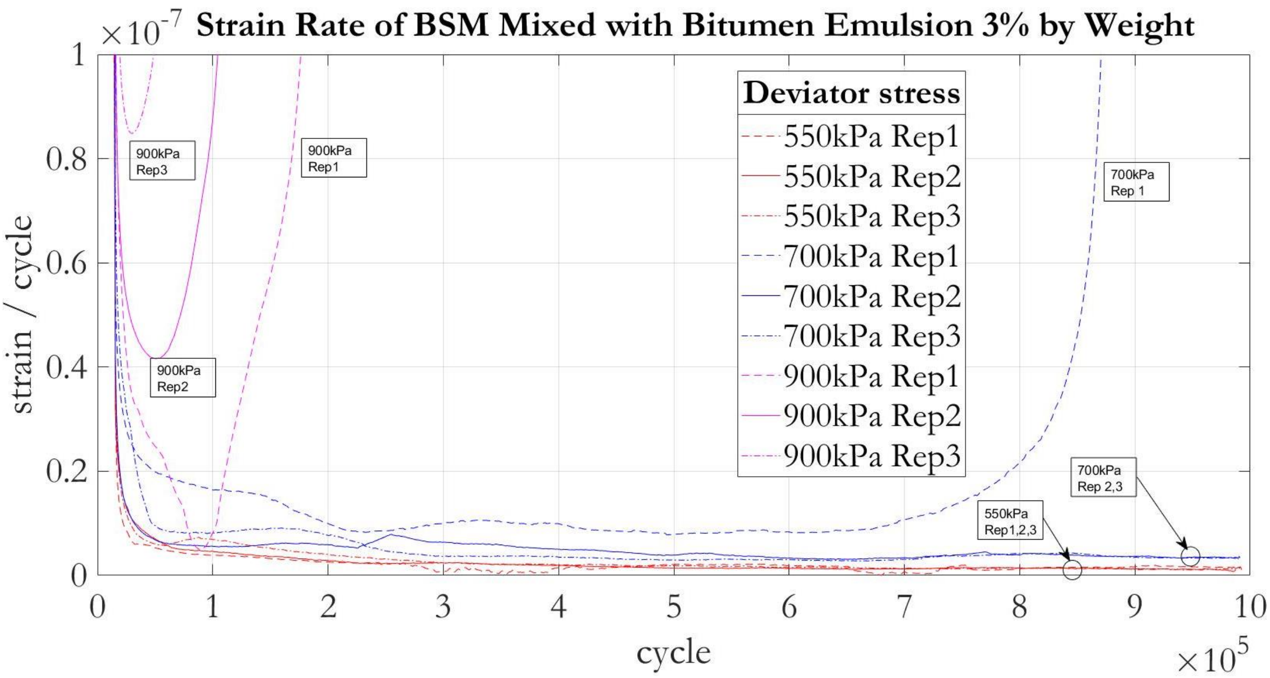

Fig. 8. Relationships between the number of repeated load cycles and the permanent strain rate of the 3% BSM specimens.

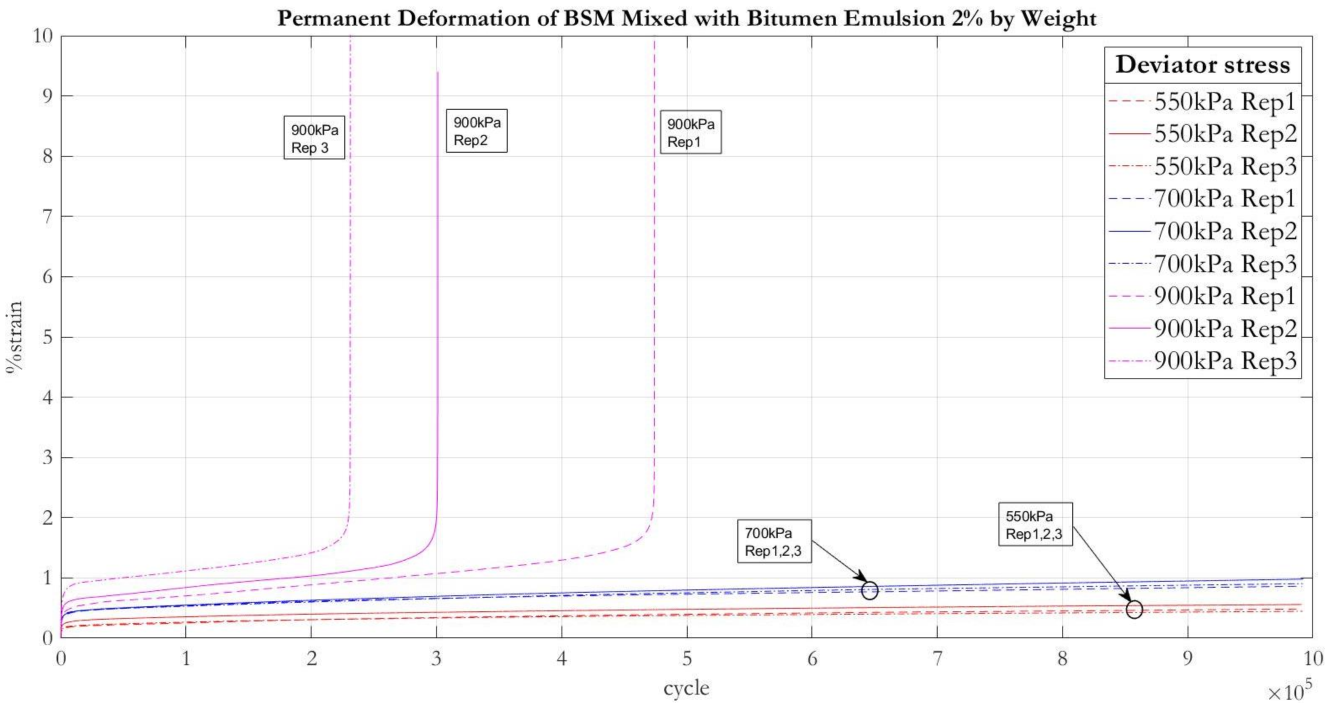

Fig. 9. Relationships between the number of repeated load cycles and the accumulated permanent strain of the 2% BSM specimens.

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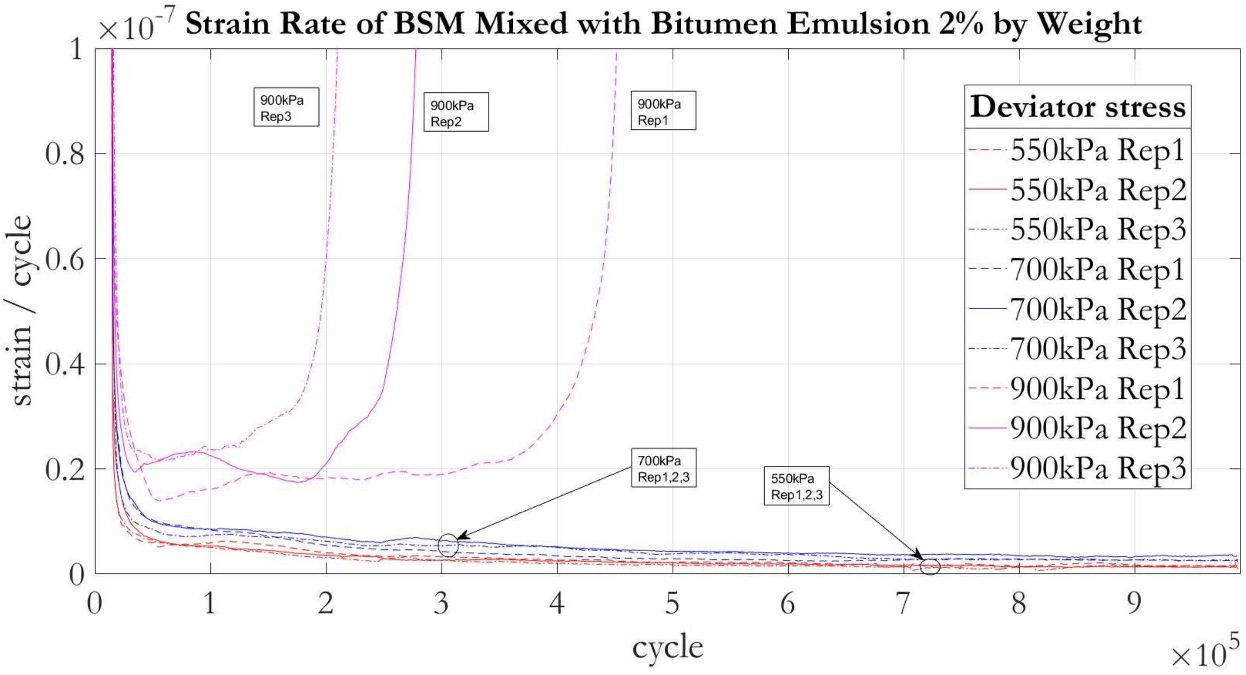

Fig. 10. Relationships between the number of repeated load cycles and the permanent strain rate of the 2% BSM specimens.

Table 4. Effects of different repeated load levels on the 3% BSM specimens.

Specimen no.	Deviator stress (kPa)	Minimum strain rates (micro strain/cycle)	Average minimum strain rates	Standard deviation of minimum strain rates	Number of cycles at the minimum strain rates	Number of cycles at the end of the test
1	900	0.005	0.044	0.040	88,500	208,594
2		0.042			50,500	134,821
3		0.085			29,500	89,370
1	700	0.008	0.0047	0.003	494,500	893,644
2*		0.003			640,500	1,000,000
3*		0.003			663,500	1,000,000
1*	550	0.001	0.001	0.00	680,500	1,000,000
2*		0.001			985,500	1,000,000
3*		0.001			941,500	1,000,000

*The specimens did not exhibit incremental collapse at 1,000,000 cycles of repeated loads.

Table 5. Effects of different repeated load levels on the 2% BSM.

Specimen no.	Deviator stress (kPa)	Minimum strain rates (micro strain/cycle)	Average minimum strain rates	Standard deviation of minimum strain rates	Number of cycles at the minimum strain rates	Number of cycles at the end of the test
1	900	0.014	0.0177	0.004	56,500	474,237
2		0.017			176,500	301,106
3		0.022			62,500	231,297
1*	700	0.002	0.0023	0.0006	635,500	1,000,000
2*		0.003			896,500	1,000,000
3*		0.002			989,500	1,000,000
1*	550	0.001	0.001	0.00	781,500	1,000,000
2*		0.001			989,500	1,000,000
3*		0.001			815,500	1,000,000

*The specimens did not exhibit incremental collapse at 1,000,000 cycles of repeated loads.

5.1. 3% BSM

For 3% BSM, the first three specimens were subjected to deviator stress at 900 kPa. All three specimens exhibited the incremental collapse pattern at early number of repetitions below 210,000 cycles. For the second group of three specimens, the applied deviator stresses were at 700 kPa. There was one specimen, replicate no. 1, exhibited plastic shakedown pattern until about 500,000 cycles. Beyond that, its strain rate started to increase at high degree polynomial, and it collapsed at 893,644 cycles. However, the other two replicates did not express excessive permanent strains above 1% permanent strain until 1 million cycles. Their strain rates remained at lower than 0.1x10-7 strain per cycle after a few 100,000 cycles. For the third group, the deviatoric stresses were at 550 kPa. All three specimens exhibited plastic shakedown patterns with the permanent strains below 0.6% at 1 million cycles. Their permanent strains and permanent strain rates were comparatively lower than those of specimens experiencing 700 kPa stress level. It seems that they became elastic shakedown at the end of the tests.

5.2. 2% BSM

For 2% BSM, the same three deviatoric stress levels of 900, 700, 550 kPa were applied to three groups of specimens. At 900 kPa, the three specimens exhibited excessive permanent strains and went to collapse between 200,000 – 480,000 cycles. At 700 kPa, all three specimens remained in plastic shakedown pattern without incurring the incremental collapse at 1 million cycles. At 550 kPa, all three specimens also exhibited plastic shakedown and became stable as elastic shakedown pattern at the end of 1 million cycles. The permanent strains were also lower than 0.6% at 1 million cycles At 550 kPa stress level, the permanent strains and permanent strain rates were comparatively lower than those of specimens at 700 kPa stress level.

It was clear that both 2% and 3% BSM specimens exhibited the shakedown behavior in response to the magnitude of the deviator stress similar to findings in those past studies [11-13, 19].

A closer look at the relationships between the permanent strain rates and the number of repetitions as shown in Figs. 8 and 10 revealed that during an incremental collapse stage, the permanent strain rates would drop to a lowest rate then rose exponentially. Conversely, a fall in the permanent strain rates followed by an indistinct growth indicated the plastic shakedown pattern of permanent deformation. These findings were consistent with those reported in Werkmeister [19]. An interesting finding can be seen from Tables 4 and 5 that once the minimum strain rates decreased to 0.004 microstrain/cycle or lower, they became stable in the plastic shakedown state.

Another issue worth discussing is whether different bitumen contents contribute to variation in permanent deformation behavior. The results of unconfined compressive strength test in Table 2 indicated that the 3% BSM was slightly stronger than the 2% BSM. However, the findings of this study suggests that the responses to repeated loads of the 3% and the 2% BSM specimens did not differ significantly. One implication of this result is that a monotonic loading test may not be the most accurate method for comparing resistance of BSM to repeated loads. The foregoing discussion is supported by the findings reported in Ebels [11]. Despite the difference in the bitumen content, Ebels similarly found that the BSM specimens entered a steady state with low accumulated permanent deformation at the minimum strain rate of 0.001 microstrain/cycle and underwent rapid collapse during the last stage of repeated load exposure.

In the light of such congruous empirical evidence, it is logical to contend that considerations in the application of BSM to pavement works may stay at one simple principle. That is, the determination of the thickness of a pavement structure should ensure that the vertical stress exerted will not exceed the plastic shakedown limit of the under laid BSM material, thereby preventing accumulated permanent deformation leading to collapse in the long run.

6. Summary and Conclusions

The present study evaluated the ability to resist permanent deformation of a BSM. To achieve this, the EN13286-7 permanent deformation test method were conducted by applying three levels of continuous haversine repeated deviatoric stresses: 550 kPa, 700 kPa, and 900 kPa on the BSM specimens. The BSM specimens composed of RAP and RCR at the ratio of 25:75. For stabilizing agents, 1% cement and CSS-1h emulsion of 2% and 3% by weight of the mixtures were applied in the mixture.

According to the findings, at the highest deviatoric stress of 900 kPa, all BSM specimens exhibited the incremental collapse pattern, suggesting that the BSM were not able to carry this stress level. At 700 kPa, most of the BSM specimens exhibited the plastic shakedown pattern with long-term permanent strain limit at 1%. However, a collapse at late number of repetitions was possible.

At 550 kPa, all BSM specimens at this stress level exhibited the stable plastic shakedown pattern and experienced the permanent strains lower than 0.6% strain at one million cycles. This permanent strain level should be considered favorable in pavement application.

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