DCT 233

# RAP Methodology: Previous Experimental Results and Creep Behavior with Geocell Reinforcement

**By**

**BUSBY ATTOH-OKINE**

**Department of Civil and Environmental Engineering**

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**Delaware Center for Transportation**
**University of Delaware**
**355 DuPont Hall**
**Newark, Delaware 19716**
**(302) 831-1446**

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*Delaware Center for Transportation*
*University of Delaware*
*Newark, DE  19716*
*(302) 831-1446*

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## Introduction

Pavement Rehabilitation is a work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy, for a minimum period of 10 years. This might include the partial or complete removal and replacement of portions of the pavement structural section.

Pavement rehabilitation is a major activity for all highway agencies and has several consequences on agency resources and traffic disruptions because of extensive and extended lane closures. The traffic volumes on the primary highway system, especially in urban areas, have seen tremendous increases over the last 20 years, leading in many instances to earlier-than-expected failures of highway pavements. The aging of the Interstate highway system and other primary systems built during the 1950s and 1960s has resulted in the expenditure of a large portion of highway funds on pavement rehabilitation. Efforts continue to be made to develop techniques and procedures that will result in cost-effective and longer-lasting pavement rehabilitation to serve the nation's highway system well into the 21st century.

Pavement maintenance describes all the methods and techniques used to prolong pavement life by slowing its deterioration rate. The performance of a pavement is directly tied to the timing, type and quality of the maintenance it receives. Pavement maintenance includes crack seals, fog seals, slurry seals, and bituminous surface treatments.

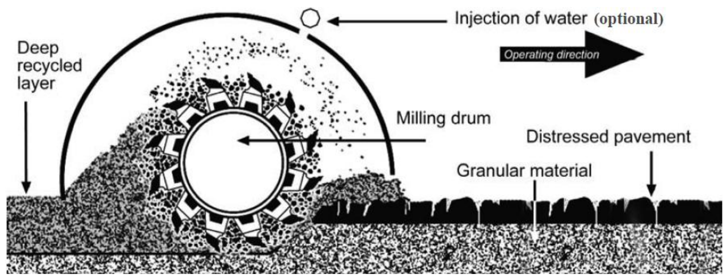
Although maintenance can slow the rate of pavement deterioration, it cannot stop it. Therefore eventually the effects of deterioration need to be reversed by adding or replacing material in the existing pavement structure. This is called rehabilitation. Rehabilitation options depend upon local conditions and pavement distress types but typically include HMA overlays, Hot In-Place Recycling (HIPR), Cold In-Place Recycling (CIPR).

Pavement recycling and reclaiming is another important process for rehabilitating asphalt pavements. Construction equipment and materials have greatly evolved over the last few years to allow for low-cost, in-place recycling and reclaiming of asphalt pavements**.** The use of reclaimed asphalt pavement (RAP) helps to conserve natural resources and land needed for disposal of these materials. Since the mid-1970s, tens of millions of tons of RAP have been used to produce recycled hot mix asphalt (HMA) around the country. The use of RAP has evolves into routine practice in many areas around the world. In the United States, the FHwA reported 73 of the 91 million metric tons of asphalt pavement removed each year and during resurfacing and widening projects are reused as part of new roads, roadbeds, shoulders, and embankments (FHWA, 2002).

## Definition and objective of RAP

Reclaimed asphalt pavement (RAP) is the term given to removed and/or reprocessed pavement materials containing asphalt and aggregates. These materials are generated when asphalt pavements are removed for reconstruction, resurfacing, or to obtain access to buried utilities. When properly crushed and screened, RAP consists of high-quality, well-graded aggregates coated by asphalt cement.

Asphalt pavement is generally removed either by milling or full-depth removal. Milling entails removal of the pavement surface using a milling machine, which can remove up to 50 mm (2 in) thickness in a single pass. Full-depth removal involves ripping and breaking the pavement using a rhino horn on a bulldozer and/or pneumatic pavement breakers. In most instances, the broken material is picked up and loaded into haul trucks by a front-end loader and transported to a central facility for processing. At this facility, the RAP is processed using a series of operations, including crushing, screening, conveying, and stacking.




Note: depending on the in-situ moisture content, water may be injected in the mixing drum during pulverization.

Fig. 1. Inside the Pulverizer (Flexible Pavement Rehabilitation Using Pulverization, CDOT 2008)

Fig. 2. Pulverizer (Flexible Pavement Rehabilitation Using Pulverization, CDOT 2008)

The recycling of RAP is typically achieved by adding a rejuvenating agent into the aged pavement mix. Using this process, the chemical composition of the aged binders coating the aggregates in the RAP mix can be adjusted, leading to improvement of the physical and performance properties of the rejuvenated binders.

The recycling of existing damaged asphalt pavement materials produces new pavements with considerable savings in material, money, and energy. Aggregate and binder from old asphalt pavements are still valuable even though these pavements have reached the end of their service lives. They have been used with virgin resources to produce new asphalt pavements, proving to be more both economical and effective in protecting the environment.

Although the majority of old asphalt pavements are recycled at central processing plants, asphalt pavements may be pulverized in place and incorporated into granular or stabilized base courses using a self-propelled pulverizing machine. Hot in-place and cold in-place recycling processes have evolved into continuous train operations that include partial depth removal of the pavement surface, mixing the reclaimed material with beneficiating additives (such as virgin aggregate, binder, and/or softening or rejuvenating agents to improve binder properties), and placing and compacting the resultant mix in a single pass.

RAP also guarantees some superior properties. Some researchers have found the stiffness of modulus of recycled mix to be than virgin mix. Similarly, the indirect tensile strength of recycled mix is found to be satisfactory or better. In general, recycled mix has a greater resistance to rutting than virgin mix. (Refer to "Pavement design with central plan hot-mix recycled asphalt mixes.")

Using RAP is environmentally friendly. It helps to conserve natural resources and land needed for disposal of these materials. Increasing the percentage of the RAP and improving the quality of recycled mixtures will facilitate the further utilization of the RAP; however, reliable figures for the generation of RAP are not readily available from all state highway agencies or local jurisdictions.

## Previous experimental results

-Stiffness and fatigue behavior.

Beam fatigue test is one of the tests that can characterize the fatigue behavior of the mix. Aravind and Das (2006) performed an experiment that a binder content of 5.5% is chosen for fatigue testing for all types of mixes. Constant strain amplitude fatigue testing is carried out to characterize fatigue behavior of recycled mixes. For this purpose, fatigue testing machine fabricated (Ghosh, 2002) at Transportation Engineering Laboratory, IIT Kanpur was used (Aravind, 2005). It consists of a motor, cam, load cell, LVDT, beam holding arrangement and a data acquisition system. Motor is used to apply sinusoidal loading on the specimen. The cam arrangement is used to impart displacement to the specimen, which in turn varies the strain in the beam being tested. A load cell is used in measuring the load applied and a LVDT is used to measure displacement. Since constant strain amplitude fatigue test is carried out, the maximum displacement is kept constant for the entire period of testing on a particular beam. The beam holding arrangement is used to fix the beam rigidly in place. A photographic view of the setup is given in Fig. 3.

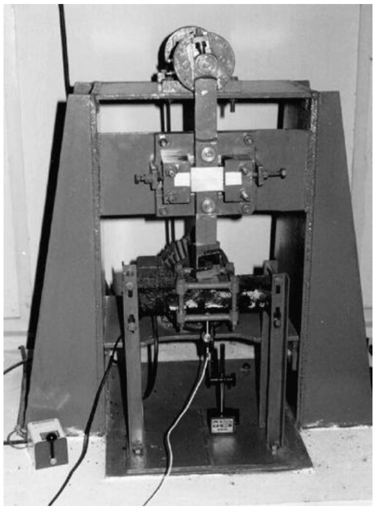


Fig. 3. The fatigue test set-up (Aravind and Das, 2006)

Asphalt mix is prepared and poured into the mold (length 380 mm, breadth 78 mm) in three layers and it is compacted with 175 blows with standard Marshall hammer. This number of blows is arrived at by comparing beam density with the average density of standard Marshall specimen at same binder content. These beams are tested for fatigue performance after 24 h of curing. Each beam is fixed to the fatigue testing machine and sinusoidal loading is applied. Load corresponding to half the initial load is noted as failure load and corresponding number of repetitions is also noted. Similar procedure is repeated for other test samples at different strain levels to develop the fatigue curve. Temperature during test is maintained 30 ± 2 °C. A general regression based fatigue equation can be expressed as

$$N = k_0 \left( \frac{1}{\varepsilon} \right)^{k_1} \tag{2}$$

where N = number of load repetitions the beam can sustain till failure, $\varepsilon$ = tensile strain, $k_0$ and $k_1$ are regression coefficients. Fig. 3 presents the fatigue curves of the various mixes. The regression coefficients of the fatigue equation (Eq. (2)) as well as the average initial stiffness values for all these types of mixes are presented in Table 1.

Table 1. Regression coefficients and initial stiffness of different mixes (Aravind and Das, 2006)

| Mix type | Number of samples tested | Regression coefficients | | $R^2$ | Average initial stiffness (MPa) |
|---|---|---|---|---|---|
| | | $k_0$ | $k_1$ | | |
| Virgin mix | 10 | 2.8150 | 1.3355 | 0.91 | 1477 |
| S1-T1 | 9 | 0.5647 | 1.5458 | 0.92 | 1711 |
| S1-T2 | 9 | 0.0631 | 1.8755 | 0.92 | 1723 |
| S2-T1 | 10 | 0.7872 | 1.3573 | 0.94 | 773 |
| S2-T2 | 10 | 0.1951 | 1.6928 | 0.78 | 738 |

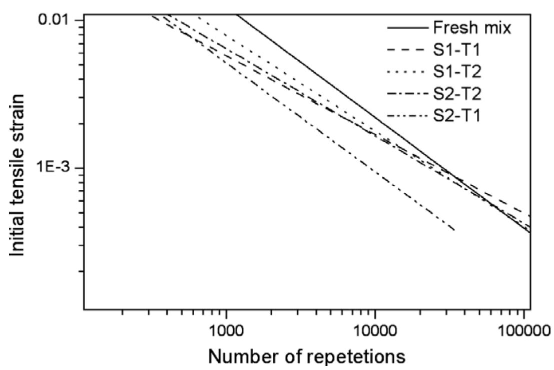



Fig. 3. Fatigue performance of different mixes (Aravind and Das, 2006)

From Fig. 3 it can be seen that at lower strain levels, the fatigue lives of the recycled mixes are better or similar to that of virgin SDBC mix. However, at higher strain level, the opposite trend is observed. Fatigue performance of S2-T1 is observed to be poorest of all other mixes. The stiffness values of Sample 1 recycled mixes are observed (refer Table 1) to be higher than virgin mix. For Sample 2 it is observed to be reverse.

Widyatmoko (2008) performed an experiment in order to assess the variation of stiffness of the materials with temperature, the Indirect Tensile Stiffness Modulus (ITSM) test was carried out on 10 unaged samples for each mixture, at 10, 20 and 30 °C. The ITSM testing was carried out under the standard conditions of 5 lm target horizontal deformation, an assumed Poisson's ratio of 0.35 and a rise time of 124 ms (British Standard Institution, 2004). Fig. 4 shows a comparison between the stiffness values of 20 mm nominal aggregate size Asphaltic Concrete Wearing Course (ACWC) and 28 mm nominal aggregate size Base Course (ACBC) mixtures with and without RAP, at three different test temperatures. Fig. 4 indicates that the mixtures containing RAP, but apart from those of ACBC containing 10% and 30% RAP, tend to have lower stiffness than equivalent mixtures without RAP. This trend would be expected since softer bitumen and/or rejuvenating agent were used in the mixtures containing RAP; however these stiffness values were at least similar to, or greater than, that of a laboratory manufactured dense asphalt with 100 pen bitumen which was reported to have an initial stiffness (at 20 °C) ranged from 1,000 to 2,500 MPa (Widyatmoko, 2002). Virgin 80/100 pen bitumen (which is one grade softer than the 60/70 pen bitumen conventionally used for the parent asphalt materials), with or without added rejuvenating oil, was used for the manufacturing of the recycled mixtures, and it should be noted that if the grade of the added virgin bitumen remains unchanged (from, in this case 60/70 pen), recycled asphalt mixtures would be anticipated to have higher stiffness.

Fatigue resistance was assessed in the NAT by using a controlled load Indirect Tensile Fatigue Test (ITFT) (British Standard Institution, 1995). The applied level of loading (hence, horizontal stress) was adjusted for individual specimens to generate lives to failure generally between 1000 and 100,000 cycles. As fatigue is a phenomenon normally associated with in-service conditions, the laboratory manufactured samples were artificially aged prior to fatigue testing (by conditioning at 85 °C for 120 h, British Board of Agreement Highway Authorities Product Approval Scheme, 2000), and the test was only carried out on aged samples. Each set of data comprised ITFT results from ten 100 mm diameter core samples. For the analysis of the ITFT data, initial tensile strains can be determined from an analysis of the stress distribution within the sample during testing. This enables a log-strain to log-life relationship to be plotted, and on this basis the results tend to conform to a single, linear relationship. Comparisons between the fatigue resistance of ACWC and ACBC mixtures with and without RAP are shown in Fig. 5, which demonstrates that the laboratory fatigue resistance of the recycled mixtures appears to be at least similar to, or better than, those of the control mixtures manufactured without RAP. The fatigue performance appears to be improved with an increased proportion of rejuvenating binder in the mixture (consequently, at higher RAP contents).

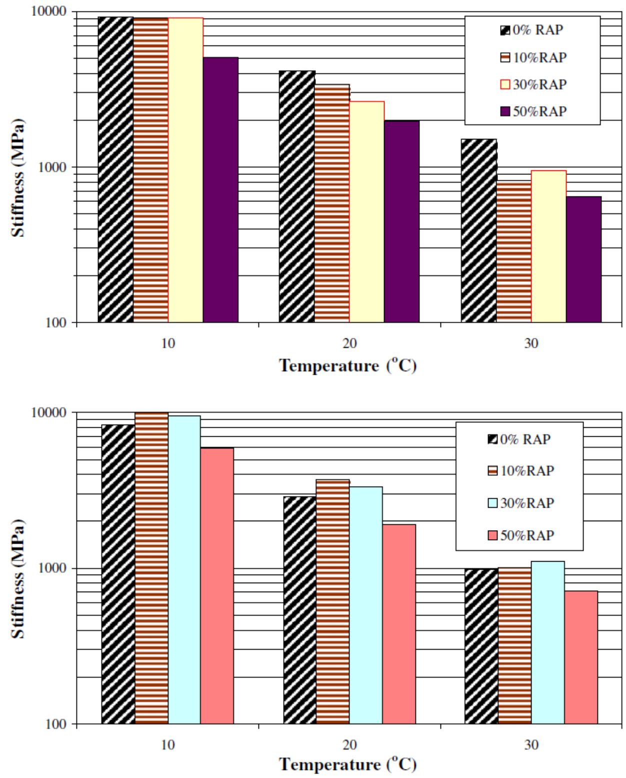



Fig. 4. Stiffness of ACWC (top) and ACBC (bottom) with and without RAP (Widyatmoko, 2008)

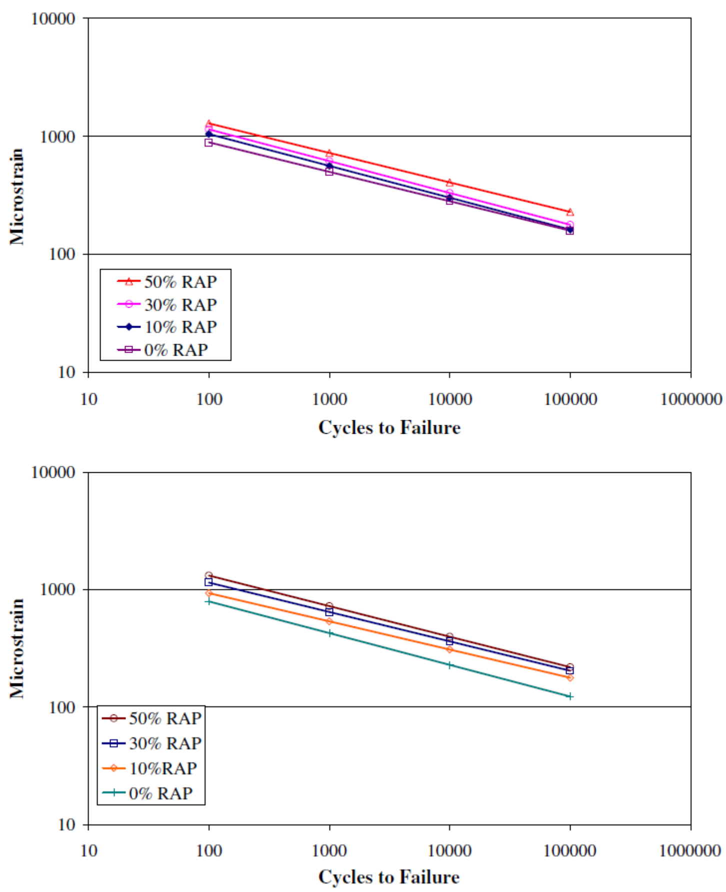



Fig. 5. Fatigue resistance of ACWC (top) and ACBC (bottom) with and without RAP (Widyatmoko, 2008)

-Resistance to rutting (Asphalt pavement analyzer: APA).

Permanent deformation, or rutting, is the accumulation of small deformations caused by densification and/or repeated shear deformations under applied wheel loads. This type of deformation is caused by consolidation, lateral movement, or both, of the bituminous mixtures under traffic. The elastic properties of bituminous mixture do no contribute to permanent deformation, while plastic properties of bituminous mixture contribute to permanent deformation under repeated loading (White et al., 2003). The resistance of bituminous mixtures to permanent deformation can be evaluated in laboratory using a number of different methods, including tests of axial compression creep, shear, and wheel-tracking.

It is well known that pavement rutting is caused mostly by the shear flow of HMA mixtures under high temperature by repeated traffic loading. This shear flow largely depends on the properties of the aggregate and the asphalt binder. This issue is one of the main concerns when utilizing RAP in mixtures containing a rejuvenator.

The asphalt pavement analyzer (APA), one of the methods to identify the rutting resistance of an asphalt mixture, has been adopted in some agencies for evaluating the rutting resistance properties of Superpave mixtures (Kandhal and Cooley, 2002; Kandhal and Cooley, 2003; Zhang et al., 2002; Martin and Parks, 2003). The APA test was used in Shen et al. (2007)'s study and carried out on cylinder samples with air void contents of 4.0±0.5%, height of 75 mm, a test temperature of 64 °C, and a hose pressure of 690 kPa. The rut depth was recorded automatically after 8,000 cycles.

Fig. 7 shows the rut average depth from six replicate samples of various mixtures containing different %RAP contents. Kandhal and Coolyey (2003) suggested tentative acceptance criteria for mixtures after 8,000 cycles using either automatic or manual measurements. For a 4% air void sample designed for the traffic level of 3 million ESALs, the acceptable rut depth is 8.0 mm for automatic measurements. In general, the rut depths of the mixture containing Source C RAP are less than that of the virgin mixture. Even for the mixture. Even for the mixture containing 38% of the RAP with softer binder, the maximum rut depth observed was still much less than the virgin mixture. The rut depths of the mixtures containing the rejuvenator are much less than those mixtures made with softer binder. It was also found that when lower RAP percentage (i.e. 15%) was used, the difference between the rut depth of the mixtures containing RAP with a rejuvenator and with a softer binder was less than the mixtures made with a higher RAP percentage (e.g. 38%)

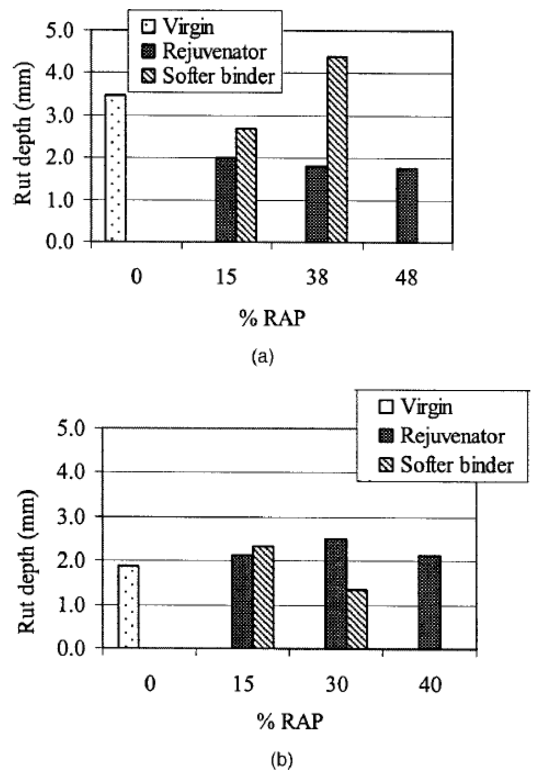



Fig. 7. Rutting depth of the Superpavement mixtures containing various RAP contents for aggregate sources: (a) C; (b) L (Shen et al., 2007)

The control mixtures containing Aggregate Source L showed a very small rut depth value. For mixtures containing 15% RAP, less difference was observed in rut depth of specimens made with a rejuvenator compared to that of a softer binder. Whereas, when a high percent of 30% RAP was used, the rut depth of the recycled mixtures using the softer binder was higher than those using rejuvenator. However, all of the rut depths were smaller than the tentative criteria value. When 40% RAP was used, the rut depth values of the recycled mixtures with rejuvenator were still acceptable.

The rut depths of the mixtures using rejuvenator were, in most cases, smaller than those using softer binder, specifically when a higher amount of RAP in incorporated. A higher percent of RAP can be incorporated by using a rejuvenator than a softer binder considering the APA results. Aggregate type still has an obvious influence on the rut depth, which can be seen by comparing mixtures containing Aggregate Source C and L.

Static creep test is one of the tests that can characterize rutting potential of a mix. This involves application of known amount of static load for a specified duration at constant temperature. Aravind and Das (2006) suggested recommendations made by Shell pavement design manual (Shell International Petroleum Company Limited, 1978) have been used for performing creep test. This involves loading and unloading for a period of 1 h each at a temperature of 40 °C. Schematic diagram of setup fabricated at Transportation Engineering Laboratory, IIT Kanpur

(Aravind, 2005) is shown in Fig. 8. It consists of a loading frame and a pair of dial gauges. Loading frame is designed in such a way that it transfers the load to the specimen axially. The samples are placed in between the smooth surfaced ceramic plates with the flat surfaces horizontal. The dial gauges are fixed at two places on the ceramic plates. The dial gauges are placed independent of the loading frame which helps in accurate measurement of axial deformation (Aravind, 2005).

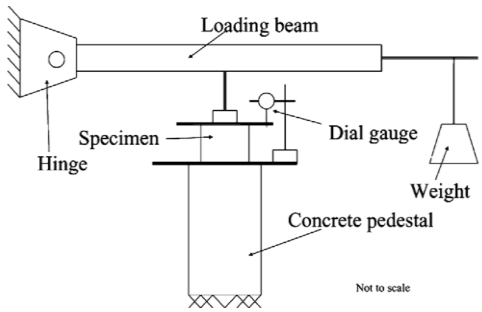



Fig. 8. Schematic diagram of creep test setup (Aravind and Das, 2006)

Each sample is tested after curing for 24 hours. Calculated amount of load is placed at the end of the loading frame such that stress of 0.1 MPa is developed in the sample. After a period of 1 hour, the load is removed. The displacements are noted at different time intervals using dial gauges over the entire period. A typical variation (for virgin mix sample with 6.5% binder content) of axial deformation with time (i.e., creep curve) is shown in Fig. 9. The recoverable strain values at different binder contents are plotted for various mixes in Fig. 10.

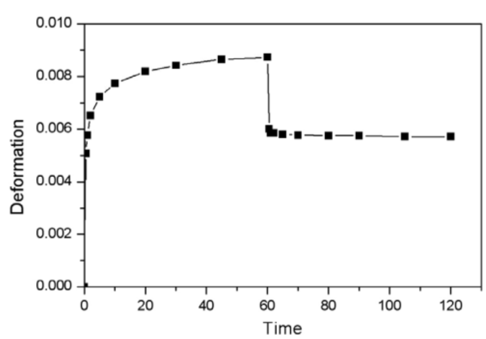



Fig. 9. Variation of deformation with time in creep test for virgin mix sample with 6.5% bitumen content (Aravind and Das, 2006)

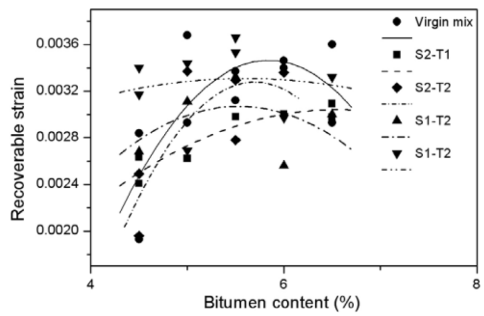



Fig. 10. Variation of recoverable strain with binder content for different mixes

(Aravind and Das, 2006)

As expected, it is seen that the recoverable strain gradually increases with the binder content and then again starts decreasing for all types of mixes. The permanent and recoverable strain components are compared in Fig. 11 for all types of mixes at particular binder content, chosen as 5.5% in the present case. It is seen that even though permanent strain component in virgin mix is less, recoverable strain in recycled mix is comparable to that of virgin mix.

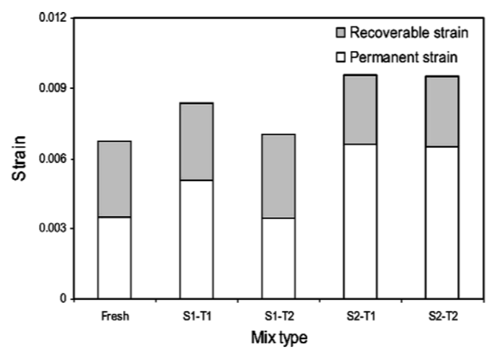



Fig. 11. Comparison of creep performance of different mixes at binder content of 5.5%

(Aravind and Das, 2006)

He and Wong (2007) performed dynamic creep test under square shape load pulse was conducted for foamed asphalt (FA) mixes in accordance with British Standard DD 226 (British Standard Institution, 1996). A load cycle consists of a stress application of 1 s duration followed by a 1 s rest period. According to BS DD 226, a standard test consists of 1800 load cycles with a maximum axial stress of 100 kPa at a test temperature of 30 °C. In the mix design, four contents of each RAP material, namely 0%, 20%, 40% and 60%, were added into the mixture respectively. The reference mix (0% RAP) for 60 pen is Group A, and the reference mix (0% RAP) for 100 pen is Group J. Materials of these two mixes are identical except for bitumens. There were totally 14 FA groups; each group was a combination of two bitumen types, two RAP materials and four RAP contents. The group codes of all mixes are listed in Table 2. After testing, dynamic creep curves of all groups were obtained. These data were depicted in Fig. 12. It can be found that there are significant differences among these curves. Each dynamic creep curve consists of two parts, one is the curve segment reflecting the densification of mixture, and the other is the line segment exhibiting the developing axial strain. Due to a relative small applied axial stress and a short loading period of 1800 cycles, failure of specimen did not occur in the test. The ultimate strains of all groups after 1800 load cycles were averaged based on three specimens' results and listed in Table 3. Fig. 13 demonstrates mean columns and error bars (I-shapes in the plot, half I-shape represents the standard deviation) of these strains. It is apparent that variance of each group's ultimate strains is not large. The results exhibit that except the ultimate strains of Group O, P and Q, all other ultimate strains roughly decrease with an increase of RAP content. This implies that the increase of RAP content may improve the resistance of FA mixes to permanent deformation.

Table 2. OMCs and DBCs of all FA groups (He and Wong, 2007)

| Bitumen | 60 pen | | | | | | | 100 pen | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| RAP | RAP #1 | | | | RAP #2 | | | RAP #1 | | | | RAP #2 | | |
| %RAP | 0 | 20 | 40 | 60 | 20 | 40 | 60 | 0 | 20 | 40 | 60 | 20 | 40 | 60 |
| Group | A | B | C | D | F | G | H | J | K | L | M | O | P | Q |
| OMC (%) | 6.30 | 6.25 | 6.11 | 6.07 | 6.31 | 6.15 | 6.09 | 6.30 | 6.25 | 6.11 | 6.07 | 6.31 | 6.15 | 6.09 |
| DBC (%) | 3.5 | 3.5 | 3.0 | 3.0 | 3.5 | 3.0 | 3.0 | 3.5 | 3.5 | 3.0 | 3.0 | 3.5 | 3.0 | 3.0 |

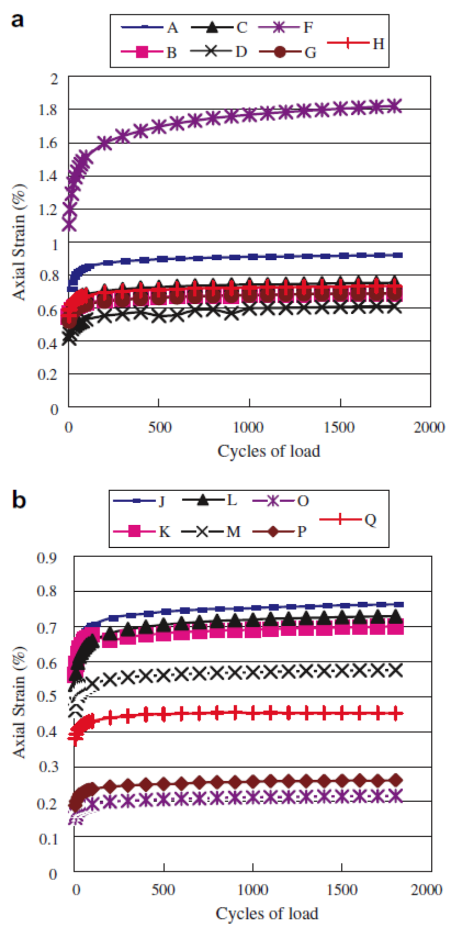



Fig. 12. Dynamic creep curves of 14 FA groups (35°C) (a) 60 pen (b) 100 pen (He and Wong, 2007)

Table 3. Summary of test results of all FA groups and hot asphalt mixes (He and Wong, 2007)

| Mix | | Ultimate strain (%) | Creep strain slope | Intercept (microstrain) | SCSM (MPa) | Air void (%) |
|---|---|---|---|---|---|---|
| FA groups | A | 0.920 | 0.162 | 8917.5 | 604.3 | 12.81 |
| | B | 0.687 | 0.147 | 6623.5 | 627.8 | 11.73 |
| | C | 0.752 | 0.170 | 7231.3 | 486.1 | 11.94 |
| | D | 0.611 | 0.259 | 5520.2 | 262.0 | 11.70 |
| | F | 1.822 | 0.130 | 13088.3 | 126.1 | 12.30 |
| | G | 0.690 | 0.189 | 6575.1 | 431.4 | 10.79 |
| | H | 0.732 | 0.138 | 7088.8 | 628.1 | 11.11 |
| | J | 0.763 | 0.149 | 7375.8 | 558.6 | 12.86 |
| | K | 0.700 | 0.132 | 6681.7 | 666.1 | 12.15 |
| | L | 0.730 | 0.161 | 7023.8 | 551.0 | 11.95 |
| | M | 0.576 | 0.094 | 5594.2 | 1136.6 | 11.59 |
| | O | 0.217 | 0.068 | 2052.2 | 1335.8 | 12.51 |
| | P | 0.261 | 0.065 | 2499.8 | 1052.3 | 11.99 |
| | Q | 0.452 | 0.080 | 4527.1 | 912.1 | 12.70 |
| Hot asphalt mixes | AC-20 | 0.336 | 0.297 | 2870.3 | 302.0 | 7 |
| | PA-10 | 0.920 | 0.654 | 8089.4 | 128.5 | 17.5 |

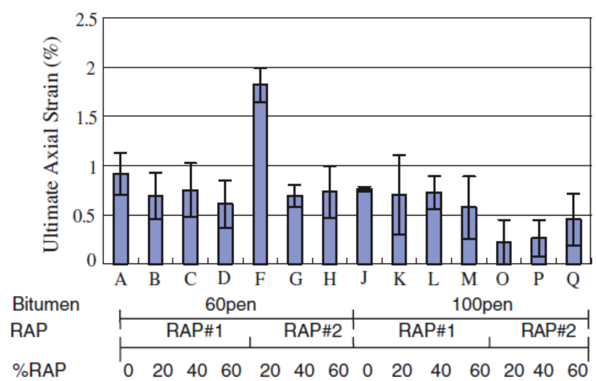


Fig. 13. Ultimate strains of 14 FA groups (35 °C) (He and Wong, 2007)

Fig. 12 shows that a curve relationship for each group between axial strain and load cycle exists. If axial strains (in microstrain) are depicted in the plot of log strain versus log load cycles (Fig. 14), all strains still exhibit a curve relationship with load cycles. In Fig. 14, however, the plot of the strains caused by the applied load after 600 cycles versus load cycles demonstrates a linear relationship with a high relationship coefficient. The slope of the last two thirds of dynamic creep curve in the log–log plot, namely creep strain slope (CSS2/3), reflects the trend of axial strain. Lager CSS2/3 indicates less resistance of FA mixes to permanent deformation. The Intercept of the fitted creep curve denotes roughly the initial axial strain of FA mixes, which reflects the permanent deformation in the densification stage. The more Intercept is, the larger the initial permanent deformation is. The initial permanent deformation of specimen compacted in laboratory, which is not caused by load cycles, is often affected by compaction method and mix’s grading. However CSS2/3 excludes the initial permanent deformation. Therefore

CSS2/3 can be used to characterize permanent deformation susceptibility of FA mixes under repeated loading. The slope of the whole log–log creep curve, namely CSS3/3, is also showed in Fig. 14.

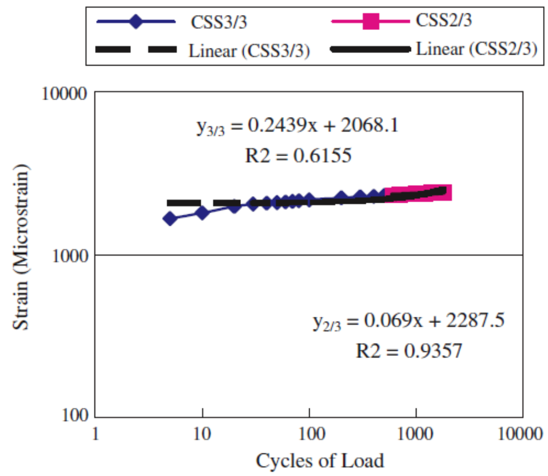



Fig. 14. Two fittings for dynamic creep curve of Group O (35 °C) (He and Wong, 2007)

Although the whole curve can be linearly fitted, its relationship coefficient is not large. Hence the CSS3/3 was not used to evaluate the susceptibility of FA mixes to permanent deformation because it contains densification. For simple expression, CSS2/3 is replaced with CSS, in microstrain per cycle, in the following context. After analysis, CSSs of all groups are summarized in Table 4. CSS of each group shown in this table is averages of three specimens. It assumes that for each bitumen, the results of mixes with 0% RAP#1 and 0% RAP#2 are identical. Fig. 15 demonstrates the column chart with error bar of CSSs versus RAP contents. Error bars of all groups indicate that results of the replicate specimens are of high deviation. The figure also shows that all CSSs of groups stabilized by 100 pen are smaller on average than those of groups stabilized by 60 pen except for result of 100 pen + 20% RAP#1. This result indicates that softer bitumen may help the FA mixes to lower their susceptibility to permanent deformation, although variation of CSS of each group caused by three factors (bitumen, RAP and RAP content) varies significantly. CSSs of FA mixes containing 100 pen and RAP#2 (Group O, P and Q) are smallest among all the groups. It implies that ageing affects the CSSs of FA mixes stabilized by 100 pen. More aged RAP material added would result the less susceptibility of FA mix to permanent deformation. Intercept (average of three specimens) of the fitted linear equation of each group is listed in Table 4.

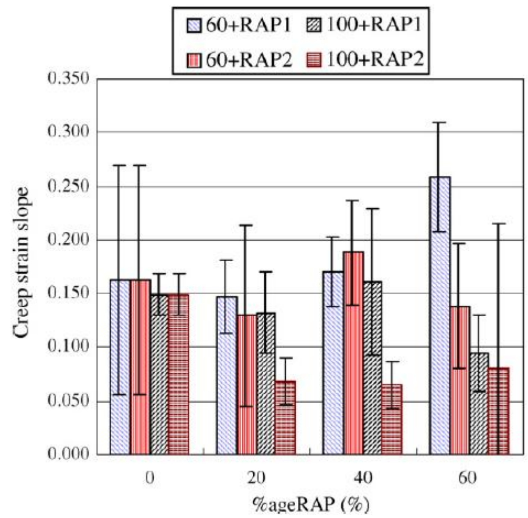



Fig. 15. CSSs of all groups versus RAP contents (35 °C) (He and Wong, 2007)

Fig. 16 is the scatter chart of all Intercepts of 14 FA Groups. It demonstrates a poor correlation between Intercept and RAP content. Although the relationship coefficient is small, this relationship exhibits that Intercept decreases with an increase of RAP content. It can be inferred that the initial axial strain decreases when RAP content increases, i.e. increasing of RAP content helps to reduce permanent deformation.

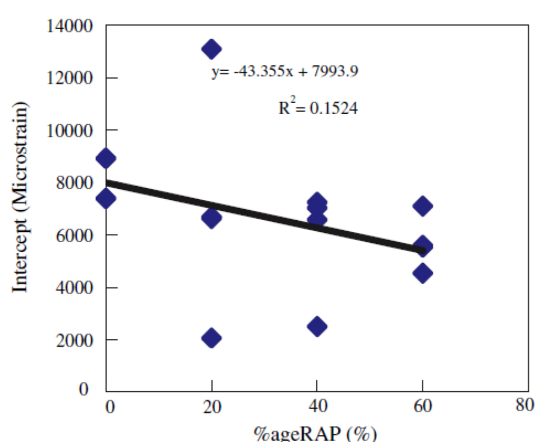



Fig. 16. Intercepts of 14 FA groups versus RAP contents (35 °C) (He and Wong, 2007)

Dynamic creep stiffness modulus is the ratio of the applied stress to the total axial strain caused to the specimen after specific load cycles at testing temperature. Disadvantage of this modulus is that it is a transient modulus, and in its calculation the total axial strain contains the initial strain caused by densification. Hence dynamic creep stiffness modulus cannot reflect the susceptibility of the permanent deformation only caused by repeated loading. For the purpose of characterizing the permanent deformation under the loading, the information in the last two thirds of dynamic creep curve is used to calculate the secant creep stiffness modulus. SCSM can be calculated by Eq. (4):

$$S_{\sec(35°C)} = \frac{\sigma_0}{\varepsilon(35°C, 1800) - \varepsilon(35°C, 600)} \tag{4}$$

where ε (35°C, 600), ε (35°C, 1800) are axial strains at 35 °C caused to the specimen after 600 load cycles and 1800 load cycles respectively, $S_{\sec\,(35°C)}$ is SCSM at 35 °C, $\sigma_0$ is the applied stress, in kPa. In this research, $\sigma_0$ = 100 kPa. SCSM is obtained from the stable developing stage of axial strain; it does not contain the information of initial axial strain caused by densification. Hence it significantly reflects the susceptibility of FA mixes to permanent deformation. SCSMs of all groups are also listed in Table 3. Each SCSM is the average of three specimens. These SCSMs range from 126 to 1335 MPa. Results of SCSMs and CSSs of 14 FA Groups are depicted in Fig. 17. There is a good correlation between these two parameters. The relationship coefficient of the fitted curve, $R^2$, is 0.70. It is apparent that CSS deceases with an increase of SCSM. So if the secant creep stiffness of FA mix is being enhanced, then its susceptibility to permanent deformation will drop as the consequence. Thus CSS and SCSM are considered to be useful parameters for evaluating the permanent deformation susceptibility of FA mixes. Foamed asphalt mixes were applied in the road sections in South Africa. It was found that Creep stiffness moduli of specimens cored from these sections varied from 58.25 to 550 MPa, and were relatively low (Muthen KM 1998). Compared with these results, all FA groups have higher moduli than those mixes in South Africa except for Groups C, D, F and G. It should be noted that SCSM does not contain densification deformation, whilst creep modulus does contain densification deformation; therefore SCSM is generally higher than creep modulus.

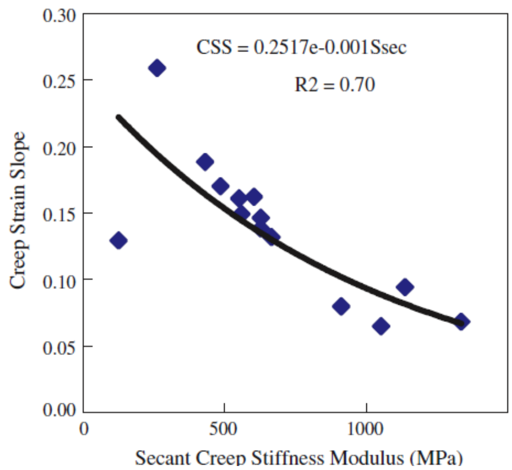



Fig. 17. Secant creep stiffness moduli ($S_{sec}$) versus CSSs (14 FA groups) (35 °C) (He and Wong, 2007)

Widyatmoko (2008) performed an experiment to assess the value of deformation resistance. The Repeated Load Axial Test (RLAT) was carried out at 40 °C in the Nottingham Asphalt Tester (NAT) under the standard test conditions of a one second square load pulse of magnitude 100 kPa, followed by a one second rest period, which has been suggested in the UK as being appropriate for most applications (British Standard Institution, 1996); the permanent viscoplastic strain, $\varepsilon_p$, was recorded after a total of 3,600 load applications were applied (or fewer if the specimen collapsed). The Wheel Track Test (WTT) was carried out under the standard test conditions of a wheel load of magnitude 520 N, which moves backwards and forwards in simple harmonic motion at 42 passes per minute (21 cycles per minute), until 45 min has elapsed, or a 15 mm rut has developed, and the permanent deformation is recorded at 5 min intervals (British Standard Institution, 1998). The WTT was carried out at 60 °C, a test temperature adopted in the UK for assessment of performance related design mixtures and premium surfacing materials for very heavily stressed sites requiring very high rut resistance (Clause NG 943 of the UK's Notes for Guidance on the Specification for Highway Works (Department of Transport)). Comparisons between the deformation resistance of 20 mm nominal aggregate size Asphaltic Concrete Wearing Course (ACWC) and 28 mm nominal aggregate size Base Course (ACBC) mixtures with and without RAP, at 40 and 60 °C, are shown in Fig. 18. The threshold values of 5.0 mm/h and 7.0 mm for the wheeltracking (WTT) rate and rut depth at 60 °C, respectively, are quoted from Note for Guidance NG943 and NG952 of MCHW2 for performance based materials for very heavily stressed sites requiring very high rut resistance (Department of Transport). The maximum RLAT strain of 2% after 3,600 applications of 100 kPa load, tested at 40 °C, is based upon experience for continuously graded UK materials, representing a threshold value above which a material can be considered as susceptible to deformation. Fig. 18 suggests that mixtures containing RAP show lower resistance to permanent deformation (i.e. greater WTT rut depth, and/or WTT rut rate, and/or RLAT strain) compared with equivalent mixtures without RAP. Overall, however, the WTT and RLAT data suggest that the resistance to permanent deformation of these recycled mixtures would be considered to be good to average, and all except one meet the UK wheeltracking requirements for very heavily stressed sites requiring very high rut resistance.

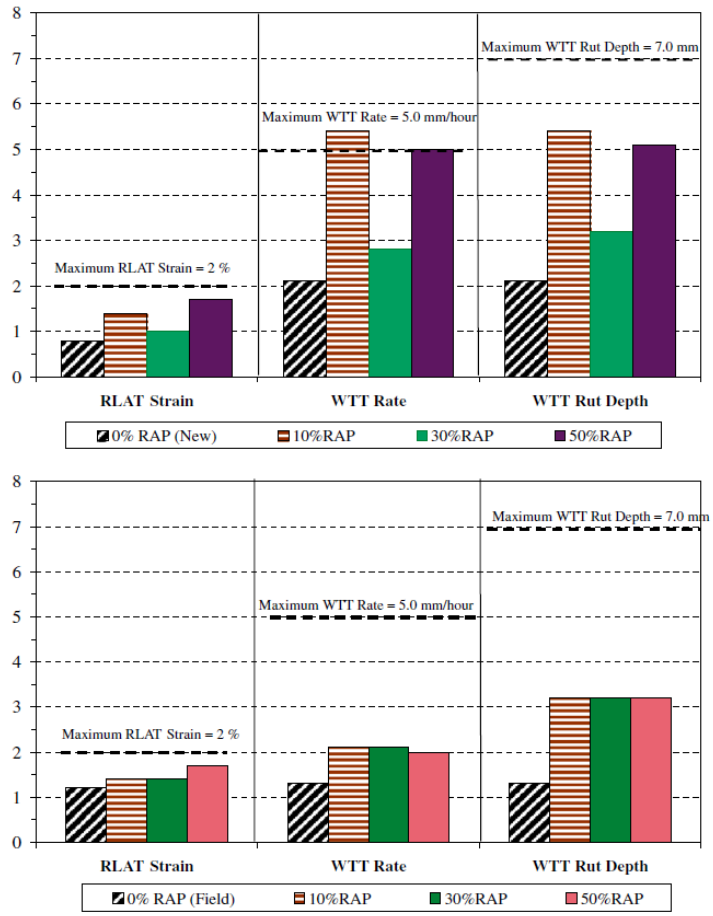



Fig. 18. Deformation resistance of ACWC (top) and ACBC (bottom) with and without RAP (Widyatmoko, 2008)

-Showing better mechanical properties (Indirect Tensile Strength test: ITS test).

The mixtures susceptibility of all mixtures was evaluated by obtaining the indirect tensile strength (ITS) of samples and calculating the tensile strength ratio (TSR) of various mixtures. Shen et al. (2007) performed an experiment with ITS test. Two sets of three samples each after being cured under different specified conditions were tested in the experiment

Fig. 19 shows the ITS average values of three samples of the mixtures incorporating Two types of aggregate referred to as C and L were selected to make virgin mixtures as control samples and mixtures containing RAP. Aggregate

Source C is granite, while Source L is gneiss. Aggregate Sources C and L in wet and dry states. Generally, it is shown that all of the Super pavement mixtures have higher wet ITS values than the standard of 0.46 MPa (65 psi) required by the SCDOT, regardless of the aggregate source, the rejuvenator agent produced higher strengths than those containing the softer binder. Furthermore, the mixtures containing RAP, in general, had higher ITS values than the corresponding virgin mixture of the same aggregate source. The mixture made with 10% more RAP source L and containing the rejuvenator still had higher strength than required by SCDOT's specifications. This trend was also true for mixtures containing Aggregate Source C. This indicated that the mixtures containing the rejuvenator produced ITS results as good as or even better than the mixes made with the soft binder. There was no apparent relationship between the percentage of the RAP incorporated in the mixtures and the ITS values. In addition, some mixtures produced higher ITS values in wet state than in dry state, especially for Aggregate Source C. This phenomenon was occasionally observed in experiments for the mixtures using hydrophobic aggregates. However, the curing of the recycled mixtures under hot water may improve the interaction of the RAP aggregate with the binders.

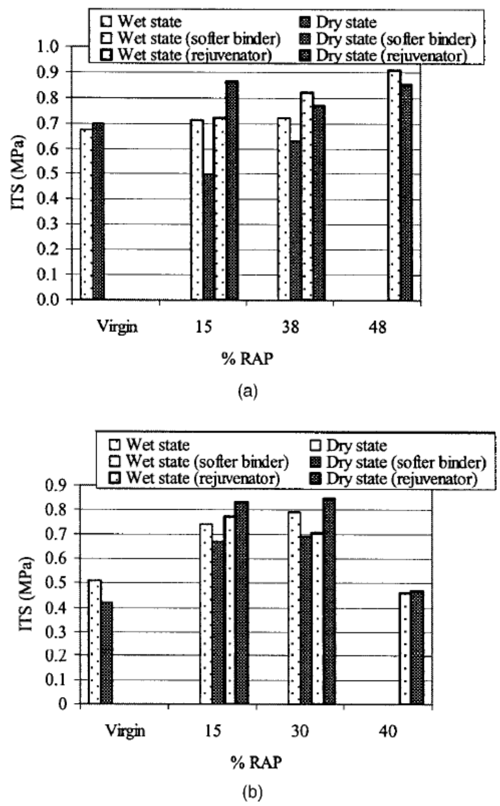



Fig. 19. Indirect tensile strength for aggregate sources: (a) C; (b) L (Shen et al., 2007)

The percent tensile strength ratio (%TSR), defined by the ITS strength in the wet state divided by that in the dry state, are higher than standard of 85% required by the SCDOT (Fig. 20) for all mixtures. For Aggregate Source C, the ratios ranged from 95.7%, the lowest for the virgin mixture and as high as 141% for the mixtures containing 15% RAP. For Aggregate Source L, the range of TSR ratio was from 83.3%, for the mixture containing 30% RAP, to 118%, for the control mixture. The data indicated that the mixtures containing RAP, in general, have better antispring properties than the virgin mixtures regardless of the aggregate type.

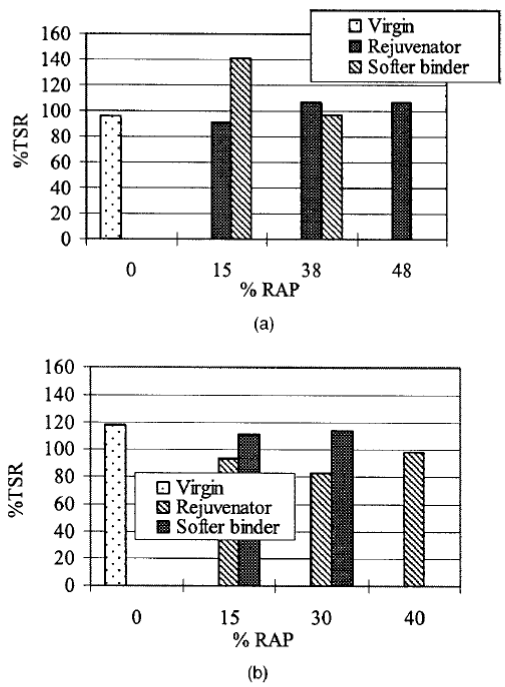



Fig. 20. %tensile strength ratio for aggregate sources: (a) Cl (b) L (Shen et al., 2007)

Base on the result, they conclude that the mixtures containing RAP have the same potential as the control virgin mixtures to provide a bearing strength and having the same type of moisture susceptibility as the virgin mixture. Test samples containing RAP did not exhibit visual signs of stripping even for the mixtures containing the highest percentage of RAP (40 and 48%) for Aggregates L and C; respectively.

### Creep behavior of RAP with geocell reinforcement

Geocell is a type of geosynthetic manufactured in a form of three-dimensional interconnected honeycomb type of polymeric cell and has been used to confine unbound aggregates for base courses of roads since the 1970s (Thakur et al. 2012). Thakur et al. (2012) conducted a series of plate loading tests to investigate the creep behavior of geocell-reinforced RAP material.

-Method

Unreinforced RAP bases, single geocell-confined base, and multi geocell-confined bases were used as samples. Loads were applied through a rigid metal plate on RAP samples by adjusting air pressure in the air cylinder. The applied vertical stress was maintained at 276 kPa or at 552 kPa for about 7 to 10 days. The displacement with time was monitored during each test. The tests were conducted at a room temperature i.e. 24°C to 26°C. The geocell used in this study was made of novel polymeric alloy (NPA) having two perforations of 100 $mm^2$ area each on each pallet, 1.1-mm wall thickness, 100 mm height, 19.1 MPa tensile strength, and 355 MPa elastic modulus at 2% strain. The tensile strength and elastic modulus were determined based on the tensile tests of geocell sheets at a strain rate of 10%/min at 23°C. The NPA is characterized by flexibility at low temperatures similar to HDPE with elastic behavior similar to engineering thermoplastic. The creep resistance properties of the NPA geocell are shown in Table 4.

The RAP and a well graded aggregate (named as AB-3) were used as the cover materials above geocell in the single geocell-confined base sections. The AB-3 aggregate was obtained from Hamm Quarry Inc., located in North Lawrence, Kansas. The grain-size distribution of AB-3 is shown in Fig. 21. The mean particle size ($d_{50}$), liquid limit, plastic limit, specific gravity at 20 $^oC$, optimum moisture content, and maximum dry density of the AB-3 aggregate were 7.0 mm, 20, 13, 2.69, 10%, and 2.08 $g/cm^3$, respectively. The standard Proctor compaction and the unsoaked CBR curves for the AB-3 are shown in Fig. 22. The CBR values were 75% at the moisture content of 7.1% and 46% at the optimum moisture content, respectively.

Table 4. Creep resistance properties of the NPA materials (Thakur et al. 2012)

| **Time (Years)** | **Stress to create 10% strain at 23°C (N/mm)** |
|---|---|
| 25 | 5.82 |
| 50 | 5.65 |
| 75 | 5.56 |

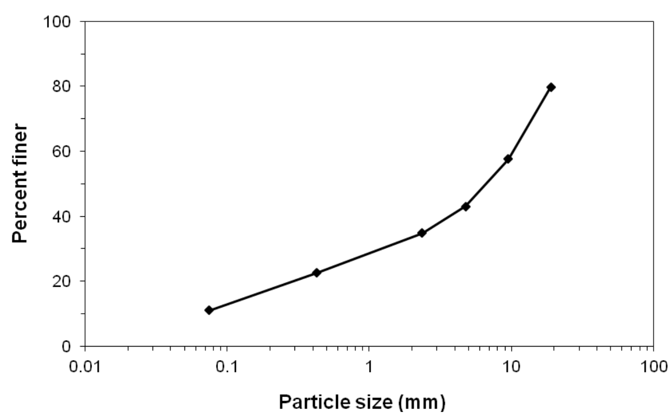



Fig. 21. Gradation curve of AB-3 aggregate (Thakur et al. 2012)

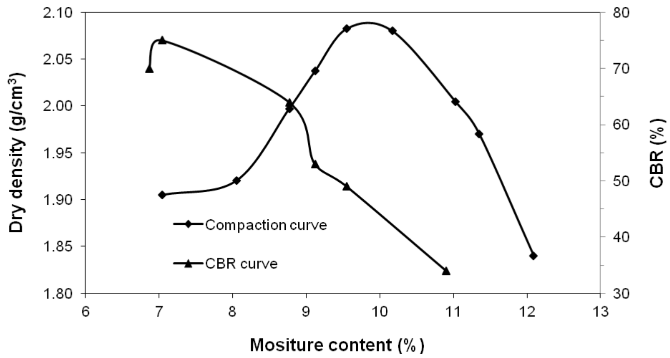



Fig. 22. Standard Proctor compaction and CBR curves of AB-3 aggregate (Thakur et al. 2012)

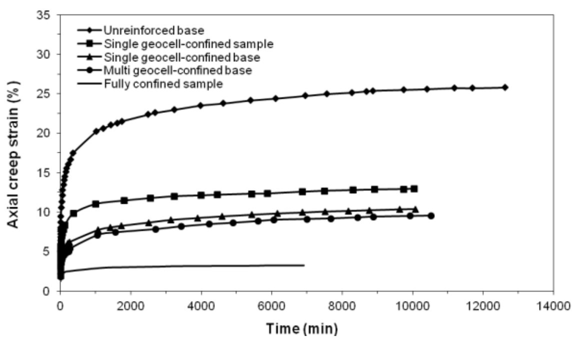
-Result

The axial strain versus time curves for RAPs at each vertical stress are presented in Fig. 23 a, b, c, and d. It is shown that the unreinforced RAP base had the largest immediate deformations within the first few minutes. The NPA geocell reduced the immediate deformation of geocell-confined RAP samples or bases by 18 to 73% as compared with the unreinforced RAP base. The NPA geocell reduced the immediate deformation of geocell-confined RAP samples or bases by 18 to 73% as compared with the unreinforced RAP base. The creep deformations were reduced by increasing the degree of confinement with the embedment of the single geocell and multi geocell in RAP. The full confinement by the rigid compaction mold reduced the immediate deformation by 81 to 86% as compared with the unreinforced base. It was also observed that the RAP samples or bases at 552 kPa deformed more compared with those at 276 kPa under the same confining condition. It was observed that the RAP crept more at the higher vertical stress and lower degree of confinement and vice versa.

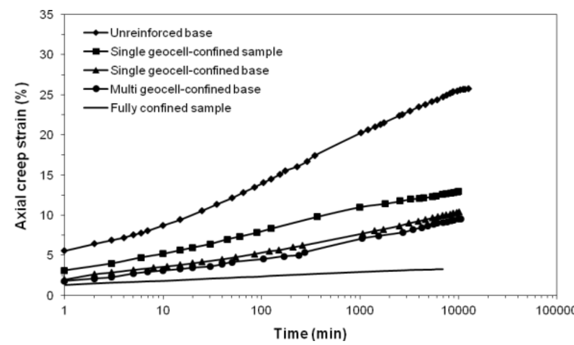
The axial creep strain versus time curves are presented in Fig. 24 to demonstrate the cover effect. The base with a RAP cover had the largest initial deformation within the first few minutes. Considering AB-3 as a rigid and low creep material (method 1), it was found that the base with the AB-3 cover had nearly the same creep deformation as that without a cover. When the total thickness of the base including the AB-3 cover and the geocell-confined RAP was used in the calculation (method 2), the axial creep strain was found to be the least because the AB-3 cover added to the thickness of the base but did not contribute much to the deformation. The AB-3 cover reduced the initial deformation by 58% as compared with the RAP cover. Therefore, to minimize the creep deformation, a low creep potential cover, such as the AB-3, should be used instead of RAP.

The creep deformation observed from 0 to 2000 minutes was termed as primary creep whereas that after 2000 minutes was termed secondary creep. The applied vertical stress was not high enough in this study to cause tertiary creep followed by creep rupture. Hence tertiary creep was not observed in this study. The rates of secondary creep were calculated from the slopes of the normal scale creep curves (Fig. 23 a and c) between 2,000 and 9,000 minutes.

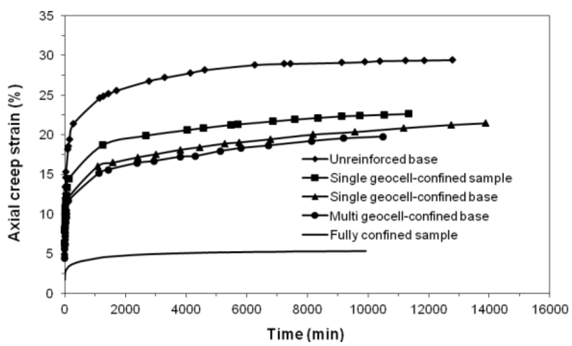
The rates of secondary creep expressed in percent per minute, under five confining conditions and at two vertical stresses, are presented in Table 5. It is clearly shown that the unreinforced RAP base had the highest creep rate, followed by the geocell confined RAP samples or bases, and the fully confined sample (Thakur et al. 2012).



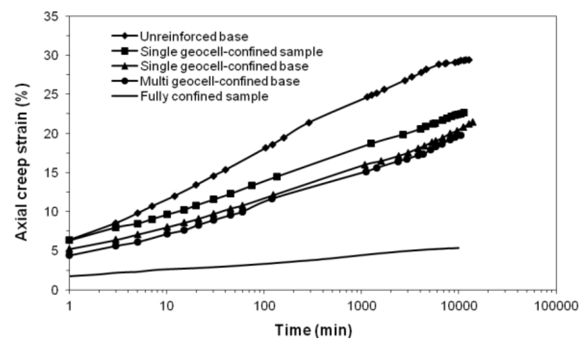

(a) at 276 kPa vertical stress (normal scale)




(b) at 276 kPa vertical stress (log scale)




(c) at 552 kPa vertical stress (normal scale)




(d) at 552 kPa vertical stress (log scale)

Fig. 23. Effect of confinement on creep behavior of RAP bases (Thakur et al. 2012)

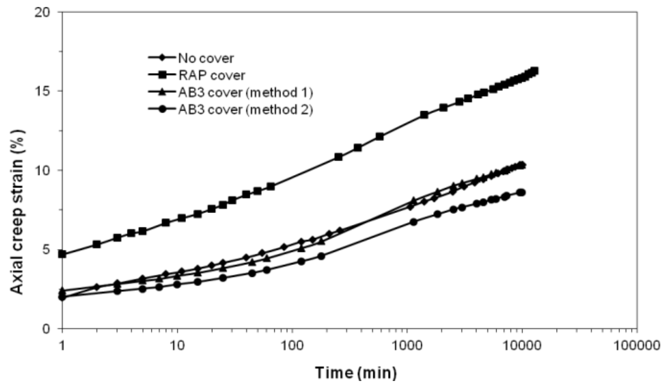



Fig. 24. Effect of cover on creep behavior of single geocell-confined bases (Thakur et al. 2012)

Table 5. Rate of secondary creep strain (Thakur et al. 2012)

| Stress Level (kPa) | Test Sample | Rate of secondary creep strain (%/min) |
|---|---|---|
| **572** | Fully confined sample | 7.12E-5 |
| | Multi geocell-confined base | 4.91E-4 |
| | Single geocell-confined base | 4.96E-4 |
| | Single geocell-confined sample | 4.42E-4 |
| | Unreinforced base | 5.23E-4 |
| **276** | Fully confined sample | 7.13E-5 |
| | Multi geocell-confined base | 2.65E-4 |
| | Single geocell-confined base | 2.71E-4 |
| | Single geocell-confined sample | 2.03E-4 |
| | Unreinforced base | 5.04E-4 |

## Conclusion

Reclaiming asphalt pavement offers cost benefits as well as competitive mechanical properties. It is also an environmental friendly method as our natural resources are preserved. Experimental studies have shown that mixes with RAP produce equivalent or improved performance to conventional asphalt mixes.

1) Stiffness and fatigue behavior

Aravind and Das (2006) performed an experiment showing the fatigue behavior recycled mix with diffrent strain level. At lower strain levels, the fatigue lives of the recycled mixes are better or similar to that of virgin SDBC mix, but at higher strain level, the opposite trend is observed.

Widyatmoko (2008) performed an experiment in order to assess the variation of stiffness of the materials with temperature.The mixtures containing RAP tend to have lower stiffness than equivalent mixtures without RAP, but it should be noted that if the grade of the added virgin bitumen remains unchanged, recycled asphalt mixtures would be anticipated to have higher stiffness.The laboratory fatigue resistance of the recycled mixtures appears to be at least similar to, or better than, those of the control mixtures manufactured without RAP.

2) Resistance to rutting

Shen et al. (2007) carried out APA test on cylinder samples with air void contents of 4.0-0.5%, and the rut depth was recorded automatically after 8,000 cycles. The rut depths of the mixtures using rejuvenator were, in most cases, smaller than those using softer binder, specifically when a higher amount of RAP in incorporated. A higher percent of RAP can be incorporated by using a rejuvenator than a softer binder considering the APA results. Aggregate type still has an obvious influence on the rut depth.

Aravind and Das (2006) suggested recommendations made by Shell pavement design manual (Shell International Petroleum Company Limited, 1978) have been used for performing creep test. They showed that even though permanent strain component in virgin mix is less, recoverable strain in recycled mix is comparable to that of virgin mix.

He and Wong (2007) performed dynamic creep test under square shape load pulse was conducted for foamed asphalt (FA) mixes in accordance with British Standard DD 226 (British Standard Institution, 1996) and the results implies that the increase of RAP content may improve the resistance of FA mixes to permanent deformation.

Widyatmoko (2008) performed an experiment to assess the value of deformation resistance. The Repeated Load Axial Test (RLAT) was carried out at 40 °C in the Nottingham Asphalt Tester (NAT) under the standard test conditions. The result suggests that mixtures containing RAP show lower resistance to permanent deformation compared with equivalent mixtures without RAP.

3) Mechanical properties

Shen et al. (2007) performed an experiment with ITS test and the mixtures containing RAP, in general, had higher ITS values than the corresponding virgin mixture of the same aggregate source. The percent tensile strength ratio

(%TSR), defined by the ITS strength in the wet state divided by that in the dry state, are higher than standard of 85% required by the SCDOT for all mixtures. The data indicated that the mixtures containing RAP, in general, have better antispring properties than the virgin mixtures regardless of the aggregate type.

4) Creep behavior with geocell reinforcement

Thakur et al. (2012) conducted a series of plate loading tests to investigate the creep behavior of geocell-reinforced RAP material.

> The NPA geocell reduced the immediate deformation of geocell-confined RAP samples or bases by 18 to 73% as compared with the unreinforced RAP base. The creep deformations were reduced by increasing the degree of confinement with the embedment of the single geocell and multi geocell in RAP. It was observed that the RAP crept more at the higher vertical stress and lower degree of confinement and vice versa.
>
> To minimize the creep deformation, a low creep potential cover should be used instead of RAP.
>
> The unreinforced RAP base had the highest creep rate, followed by the geocell confined RAP samples or bases, and the fully confined sample.

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# Delaware Center for Transportation University of Delaware Newark, Delaware 19716

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