selection technique was based on maximizing or minimizing the real functions, called objective functions, by sequentially choosing the values of real or integer variables from available subsets. In the present study, the multi-objective optimization technique was used to identify the significant aggregate subsets and binder grade combinations, which can address the common distresses on HMA pavements. The objective functions for the optimization process were the regression equations (eq.1, 2, and 3) described in the previous sections. The goals and constraints of the objective functions considered during the optimization process are presented in Table 3.

**Table 3. Constraints and Goals for Objective Functions (Rahman 2010).**

| Projects | Constraint Functions | Goals |
|---|---|---|
| **US-160** | $0 \le PG \le 1$<br>$32 \le CA1 \le 45$<br>$26 \le CA2 \le 33$<br>$15 \le NSC \le 35$<br>$CA1 + CA2 + NSC = 93$ | $NWP \ge 20{,}000$<br>$TSR \ge 80\%$ |
| **K-25** | $0 \le PG \le 1$<br>$30 \le CA1 \le 40$<br>$33 \le CA2 \le 43$<br>$15 \le NSC \le 35$<br>$CA1 + CA2 + NSC = 98$ | $NWP \ge 20{,}000$ |

The goals of objective functions for rutting and moisture damage were to minimize these distresses by maximizing the number of wheel passes (NWP) and tensile strength ratio (TSR) higher or equal to 20,000 repetitions and 80%, respectively. The constraint functions were based on the upper and lower limits of the individual aggregate subsets. The binder grade PG was considered to be continuous within 0 to 1 limit. The summation of coarse, screening, and river sand materials within the aggregate blend were 93% and 98% for US-160 and K-25 mixes, respectively. During this optimization problem, the feasible solutions generated unique values of binder grade, percentage of coarser material, screening material, and river sand content considering the limits and constraints.

The multi-objective optimization process proposed feasible aggregate and binder combinations to address all three major distresses on the asphalt pavement at each location. For US-160 mixes, binder grade PG 64-22 proved to be effective over PG 70-22. Seventeen to 22 percent river sand in the designed aggregate blend was sufficient to produce optimized design combinations instead of using 35% natural sand (allowable state practice). On the other hand, K-25 mixes with 15 to 20 percent natural sand content were found to produce the optimized mix design. Higher binder grade PG 70-22 was more effective for the K-25 aggregate source compared to PG 64-22.

## CONCLUSIONS

Optimized mix design developed for 4.75-mm NMAS Superpave mixture primarily focused on aggregate/binder combination and completely addressed the common distresses found on asphalt pavements. However, the optimum aggregate-binder combinations are different from the in-place mixes used on two projects. Investigation on RAP was outside the scope of this research study. Moreover, because of larger aggregates in RAP, it is doubtful whether RAP can be used in 4.75 mm NMAS mixture without some kind of fractionation of RAP. Again the process would add to the cost. Based on this study, the following conclusions can be made:

- Rutting performance during the Hamburg wheel tracking device tests was aggregate source specific. Effect of higher binder grade on rutting performance is inconclusive.
- The anti-stripping agent affected the moisture sensitivity test results, irrespective of natural sand content, binder grade, and aggregate source. Mixes without anti-stripping agent failed to meet the Tensile Strength Ratio (TSR) criteria specified by the Kansas Department of Transportation.
- Optimized design combinations suggested limiting the river sand content in between 15% and 20% rather than 35% (current practice) for the Kansas 4.75-mm NMAS Superpave mixture.

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# Mechanical Response of Modified Asphalt Pavements

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## Abstract

In this work, pavement cross-section as stipulated by the Indian Roads Congress code of practice for pavement design was used in the stress-analysis. Typically, two types of asphalt layers are laid over three to four layers of granular materials for pavements constructed for heavy traffic volume in India. Asphalt mixtures pertaining to these top two layers were fabricated with polymer modified and unmodified asphalts in the laboratory. These mixes were tested in the simple performance test equipment at temperature of 60 $^{o}$C under a wide range of frequencies. A four parameter Burgers' model was used for fitting the experimental data for both the asphalt layers. A two-dimensional finite element model of the pavement structure was used within the ABAQUS computing environment. The granular base, subbase and subgrade granular layers were assumed to be elastic. The pavement model was subjected to cycles of periodic loading and the stresses and strains were monitored at all the critical locations. This was used to quantify the influence of modifiers on the mechanical response of the pavement and parametric analysis was carried out to find out enhanced service life of the asphalt layers that could be achieved due to the use of high-performance materials.

**Keywords**: Viscoelasticity; Finite element analysis; Modified asphalt; Pavement engineering

## Introduction

Stress-strain analysis of an asphalt pavement structure subjected to random traffic load and varying environmental conditions is a complex task. One important factor which plays the critical role is the use of appropriate material constitutive relation in the structural model. Since the asphalt mixtures exhibit viscoelastic stress-strain behavior, identification of tractable constitutive model and using them within an appropriate computational framework becomes a critical issue. This becomes all the more important when one is interested in quantifying the beneficial effects of asphalt layers constructed with modified asphalts. Finite element modeling of pavements, if validated can be extremely useful, because it can be used directly to estimate primary response parameters without resorting to potentially costly field experiments (Helwany et al. 1998).

Different types of modeling attempts are reported in the literature as far as characterization of viscoelastic property of asphalt mixtures are concerned. For instance one can see the use of Boltzman's superpostition principle with time-dependent function (Papagiannakis et al. 1996), time dependent shear modulus functions (White et al. 1997), generalized Kelvin model at intermediate and high temperature (Elseifi et al. 2006), generalized Maxwell model (Mulungye et al. 2007) and elasto-visco-plastic model (Kettil et al. 2007) to mention a few.

Due to the complex relationship between loading type and rate on the response of asphalt pavement, issues related to quantifying the damage based on static load or dynamic load needs considerable clarification. For instance Papagiannakis et al. (1996) found the relative damage to range from 1.2 to 2.8 times depending on whether a static or dynamic load was applied. It was found that axle configurations (single axle and dual tandem) substantially influence the primary response of pavement structure and longitudinal strains are highly sensitive to tire pressure both at the top and bottom of viscoelastic layer (Helwany et al. 1998). The stress state resulting from a radial tire was found to be larger in magnitude and focused near the surface than those obtained from the traditional vertical loading conditions. These high shear stresses near surface could be an explanation for rutting failure in asphalt mixes as discussed by Novak et al. (2003). It was also found that cyclic loading on non-linear granular layers results in large deflection on subgrade than the static loading (Hadi and Bodhinayake, 2003).

Perkins and Edens (2002) introduced a parameter called traffic benefit ratio (TBR) and defined it as the ratio of vertical compressive strain on the subgrade with and without geo-synthetic reinforcement using 12.5 mm rutting as the failure criteria. This ratio was incorporated in an empirical pavement damage model and was used to explain the enhancement in pavement life.

Modification to the base asphalt is carried out to improve the rutting and fatigue cracking resistance of the material. It was found that by using styrene-butadiene-styrene polymer (SBS) modified asphalt in the top layer alone, the service life could be enhanced by 1.295 times and thickness reduction of 30 % could be achieved (Hadidy and Tan, 2009). Also use of polypropylene modified asphalt in stone matrix asphalt (SMA) was found to enhance the service life by 1.48 times and a total thickness reduction of 40 % could be achieved in SMA layer alone (Hadidy and Tan, 2009). It is also reported in the literature that decrease in thickness can also result in increased strain at the top of the layers leading to top-down cracking (Kim et al. 2009).

As seen from the above discussion, a wide variety of investigations have been carried out to characterize the stress-strain response of pavements using finite element methods. Different constitutive models for asphalt layers ranging from linear elastic to viscoelastic to visco-plastic have been used. Granular layers have been modeled either as elastic-plastic or linear elastic. The influence of different loading conditions and tire configurations have also been investigated and compared with field data. However, one of the important issues missing in most of these studies is the quantification of the benefits of using modified asphalt in all the asphalt layers. While it is well known that the use of modified asphalt for the surface course can lead to substantial structural advantage, the benefits of using modified asphalt for all the asphalt layers has not been quantified. It will also be interesting to investigate the role of asphalt layer thicknesses on the final performance of the pavement system. This work reported here is aimed at finding answers to such questions.

## Experimental Investigations

### *Materials*

SBS polymer modified asphalt (PMB70) and viscosity grade 30 (VG30) asphalt confirming to Bureau of Indian Standards, India (IS:15462, 2004 and IS:73, 2006) were used. Aggregate gradations recommended by Indian specification (MoRT&H, 2004) for the surface and binder course layers were adopted and are shown in Table 1.

Table 1.Aggregate gradations

| Sieve size, mm | Bituminous concrete (BC)-Grade 2 | | Dense bituminous macadam (DBM)-Grade 2 | |
|---|---|---|---|---|
| | Specification limits, % passing | Adopted, % passing | Specification limits, % passing | Adopted, % passing |
| 37.5 | | | 100 | 100 |
| 26.5 | | | 90-100 | 95 |
| 19 | 100 | 100 | 71-95 | 83 |
| 13.2 | 79-100 | 89.5 | 56-80 | 68 |
| 9.5 | 70-88 | 79 | - | |
| 4.75 | 53-71 | 62 | 38-54 | 46 |
| 2.36 | 42-58 | 50 | 28-42 | 35 |
| 1.18 | 34-48 | 41 | - | |
| 0.6 | 26-38 | 32 | - | |
| 0.3 | 18-28 | 23 | 7-21 | 14 |
| 0.15 | 12-20 | 16 | - | |
| 0.075 | 4-10 | 7 | 2-8 | 5 |

### *Material characterization*

Bituminous concrete and dense bituminous macadam mixes used in the surface and binder course layers were prepared at constant asphalt content of 5.25 and 4.5 %

respectively. Dynamic modulus test on asphalt mixes with modified and unmodified asphalts was carried out as per AASHTO TP62, 2007 using simple performance test equipment. Controlled haversine compressive loading, with frequency varying from 25 to 0.01 Hz was applied. The test results reported here correspond to 60 °C. All specimens were tested under unconfined condition within the linear viscoelastic range by keeping the strain in the range of 75 to 125 micro-strains. Variation in the storage and loss modulus of asphalt mixes with frequency for modified and unmodified asphalt is shown in Figure 1 and 2.

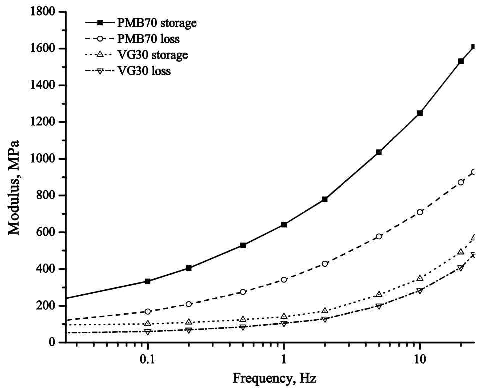


Figure 1. Storage and loss modulus variation with frequency for BC mixes

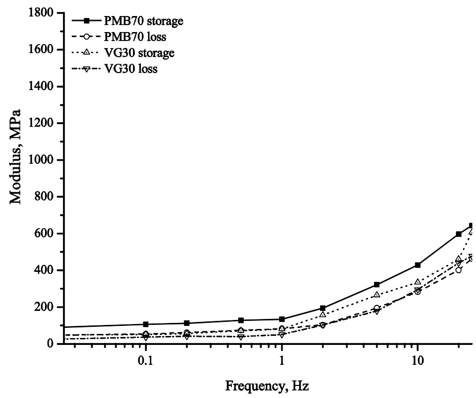


Figure 2. Storage and loss modulus variation with frequency for DBM mixes

## Flexible Pavement Composition

A five layer typical pavement structure (Figure 3) composing of asphalt and granular layer resting on a prepared subgrade of 10 % CBR for three different traffic levels as recommended by Indian Roads Congress Specification (IRC-37, 2001) is considered for the analysis. Table 2 summarizes the typical layer thickness corresponding to each traffic level.

Table 2.Flexible pavement composition for different traffic levels

| Type of layer | Thickness of each layer , mm | | | Elastic modulus , MPa | Poisson's ratio |
|---|---|---|---|---|---|
| | 50 MSA* | 100 MSA | 150 MSA | | |
| Bituminous concrete (BC) | 40 | 50 | 50 | var* | 0.35+ |
| Dense bituminous macadam (DBM) | 110 | 130 | 150 | var | 0.35+ |
| Granular base | 250 | 250 | 250 | 370+ | 0.40+ |
| Granular subbase | 200 | 200 | 200 | 170+ | 0.40+ |
| Prepared Subgrade | 500 | 500 | 500 | 77+ | 0.40+ |

*MSA= Million standard axles (traffic level), *var =Variable, + =as per IRC-37, 2001.

## Finite Element Analysis

### *Model*

Pavement structure shown in Figure 3 was modeled as two dimensional axisymmetrical using ABAQUS 6.8 finite element package. CAX4R (4-node, reduced-integration, axisymmetric, solid element) element types were used for meshing. In order to reduce the CPU time, finite element mesh was refined in the region close to the axis of application of load. A total of 7497 nodes and 7300 element were formed for typical profile of 150 MSA.

Along the axis of symmetry, shear stresses and radial displacements were constrained to zero. Bottom of the subgrade and the other end was fully restrained by providing fixed end conditions. Interface conditions were assumed to be rough. A circular loading of radius 160 mm and 650 kPa contact pressure was applied. Dynamic haversine load with rest period, in order to simulate the moving load pattern was used in the present study. A loading time of 0.1 s with an increment of 0.01 s and 0.9 s rest period was repeated for 100 cycles.

Simulations were conducted in the Vega super-cluster at IIT Madras. This is a super computer with 256 nodes with each node consisting an Intel E5472, Quad-core dual processor with 16 GB RAM. Each simulation run for 100 cycles took nearly 21200 s of CPU runtime.

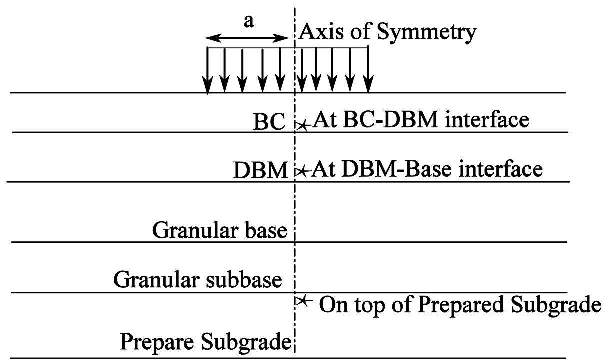



Figure 3. Pavement structure with response points and loading

***Material Models***

In this investigation, the granular materials were characterized as linear elastic materials and the material properties were taken from the Indian Roads Congress Specification (IRC-37, 2001). The asphalt layers were characterized as linear viscoelastic material. Asphalt mixes properties were modeled by assuming a linear viscoelastic Burgers' model. The constitutive relation for Burgers' model is given by:

$$\sigma + P_1 \dot{\sigma} + P_2 \ddot{\sigma} = q_1 \dot{\varepsilon} + q_2 \ddot{\varepsilon} \tag{1}$$

where, $\sigma = stress$, $\varepsilon = strain$, $P_1 = \frac{\eta_1}{R_1} + \frac{\eta_1}{R_2} + \frac{\eta_2}{R_2}$, $P_2 = \frac{\eta_1 \eta_2}{R_1 R_2}$, $q_1 = \eta_1$ and $q_2 = \frac{\eta_1 \eta_2}{R_2}$.

The viscoelastic material properties were obtained by fitting Burgers' model to the experimental data using non-linear least square regression analysis where the parameters were constrained to be positive. Time domain viscoelastic material model in ABAQUS was used in the present study. Hence, to do so stress relaxation data was generated using the calibrated Burgers' model parameters. The normalized stress relaxation data was fitted to the following relation:

$$E_r(t) = \sum_{i=1}^{N} g_i (1 - e^{-t/\tau_i}) \tag{2}$$

where, $g_i$ and $\tau_i$ are the material parameters and $E_r$ – Relaxation modulus.

The expansion of this Prony series was used in ABAQUS to input time domain material parameters. In the present study, four parameter Prony series was used to model the viscoelastic response of asphalt mixes. Material parameters for different mixes at 60 °C are tabulated in Table 3.

Table 3. Time domain viscoelastic material parameters

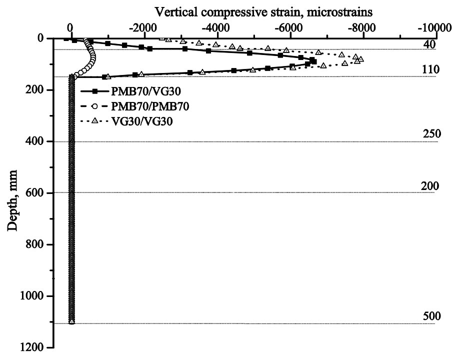
| Type of asphalt mix | Asphalt type | $g_1$ | $g_2$ | $\tau_1$ | $\tau_2$ |
|---|---|---|---|---|---|
| BC | VG30 | 0.783 | 0.217 | 0.051 | 2.804 |
| | PMB70 | 0.642 | 0.358 | 0.118 | 5.16 |
| DBM | VG30 | 0.990 | 0.010 | 0.082 | 13.437 |
| | PMB70 | 0.850 | 0.150 | 0.085 | 22.478 |

## Simulation Results and Discussions

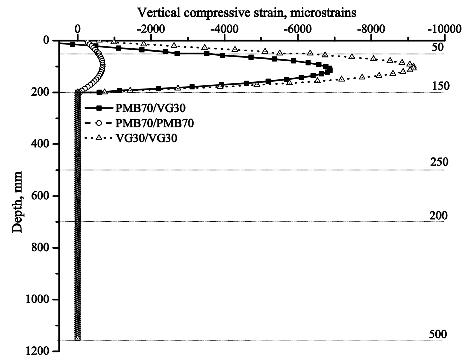
***Vertical compressive strain***

As discussed earlier, the simulation consisted of application of 0.1 s of haversine loading of 650 kPa peak magnitude followed by 0.9 s rest period. This constitutes one cycle and the pavement structure was subjected to 100 such cycles. Figure 4 shows the variation of vertical compressive strain along the depth of the pavement at the end of the application of 100 cycles. As it is seen, vertical compressive strain at the end of 100 cycles in the pavement structure with PMB70 in both asphalt layers is considerably less than that of pavement structure with VG30 and PMB70 in top layer alone. Also, the strain in granular and subgrade layers completely recover. Figure 4b and 4c shows the variation in strain for a higher traffic level, where the thickness of the asphalt layers alone was variable. It is very interesting to note that the vertical compressive strain along the pavement depth shows a maximum strain for pavement cross-sections having higher asphalt thickness. This variation depended on whether modified asphalts were used in the top two asphalt layers or not. For instance, there is no difference in the strain in the AC and DBM layers having polymer modified asphalts for 50, 100 and 150 MSA. There is a slight increase in strain as the total thickness of the asphalt base layers are increased (from 150 for 50 MSA to 180 for 100 MSA and then to 200 for 150 MSA). It is also observed that modified asphalt mixes experienced lower strains when compared to pavement structure with unmodified asphalt mixes. From this it could be ascertained that increase in the thickness of viscoelastic layer alone to imposed traffic loading demands might result in premature and early failures due to higher strain levels in the asphalt layers. These findings are similar to the results of Kim et al. (2009).

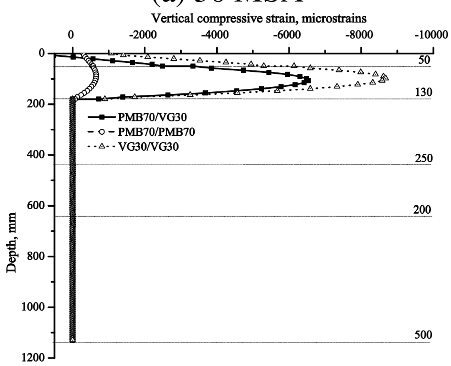
Response of pavement structure to loading and unloading can be noticed from Figure 4 (d) at 100$^{th}$ cycle (for 150 MSA). It is seen that during loading pavement structures with VG30 asphalt in both layers and PMB70 asphalt in top layer result in higher strain levels in asphalt layers when compared to pavement structure with PMB70 asphalt in both layer. It is interesting to notice that at depth of 200 mm (bottom of DBM layer) pavement structure with PMB70 in both layers recovers strain by 89.70 %, whereas pavement structures with VG30 asphalt in both layers and PMB70 asphalt in top layer alone recovers by 45.96 and 50.67 % respectively. This essentially underlines the fact that asphalt mixtures with polymer modified binders exhibit a high viscoelastic solid-like behavior and hence are able to recover the strains much faster than mixtures with unmodified binders.



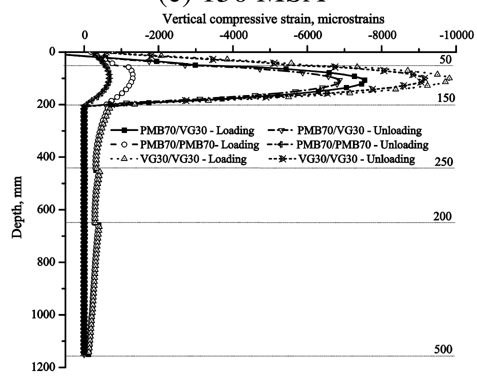

(a) 50 MSA




(c) 150 MSA




(b) 100 MSA




(d) 150 MSA

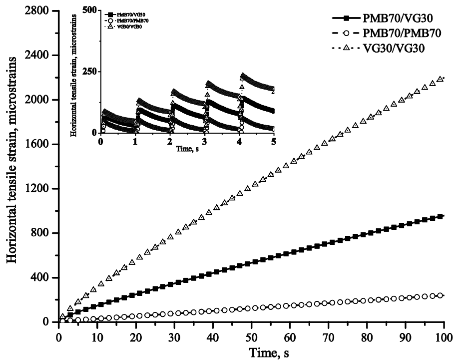
Figure 4. Variation of vertical compressive strain along the pavement cross-section

### *Horizontal tensile strain*

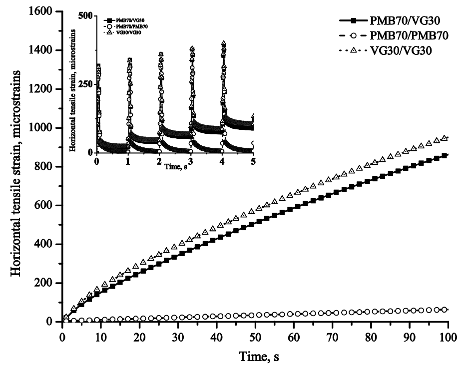
Figures 5 (a, b, and c) shows the accumulation of tensile strain at the interface of BC and DBM layer for all traffic levels (50, 100 and 150 MSA) respectively. Marked difference in the accumulated horizontal tensile strain in the surface layer of pavement structures with modified asphalt mixes and unmodified asphalt mixes was observed. The inset figure shows the loading pattern and recovery with accumulation in each case considered in the present study. Figures 5 (d, e, and f) shows the accumulated horizontal tensile strain at the DBM-granular layer interface. It is seen from these figures there exists distinct difference in the accumulated horizontal tensile strain levels at the bottom of DBM layer (interface between asphalt and granular layers) in case of pavement structure with both layers with PMB70. The trend was same irrespective of the layer thickness considered in the present study.

### Strain on subgrade

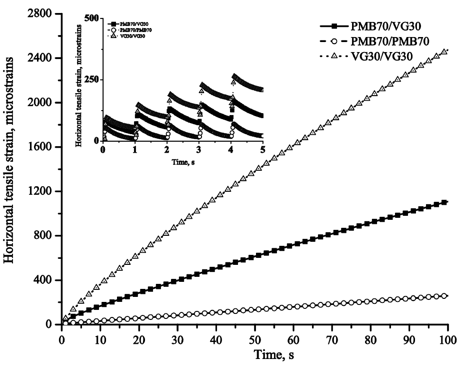
Vertical compressive strain on subgrade, which is considered to be critical for rutting in the pavement structure, is shown in Figure 6 (a, b, and c) for all traffic levels. As can be seen the maximum strain experienced by the pavement structure on top of the subgrade with modified asphalt in both asphalt layers (PMB70/PMB70) was considerably lower when compared to pavement structure with unmodified asphalt and modified asphalt in top layer alone. Use of modified asphalt in both the asphalt layers in the pavement can result in substantial reduction of strain on top of subgrade. This might lead to reduction in rutting due to the component of rutting based on subgrade material property alone.



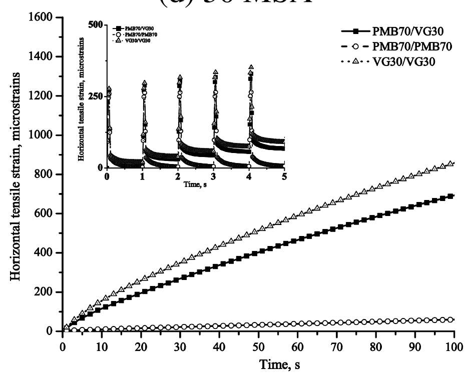

(a) 50 MSA



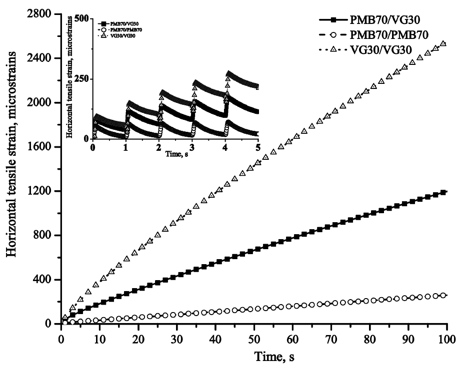

(d) 50 MSA



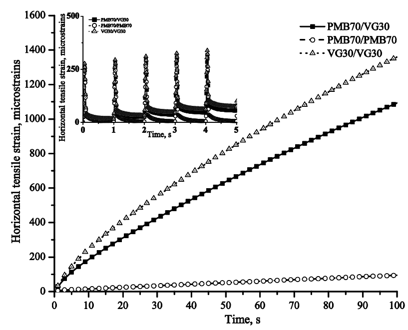

(b) 100 MSA




(e) 100 MSA




(c) 150 MSA




(f) 150 MSA

Figure 5. Accumulated tensile strain at the interface of BC-DBM and DBM-Base layers

**Quantification of Traffic Benefit Ratio**

Indian specification for guidelines and design of flexible pavements (IRC-37, 2001) relates the vertical compressive strain on the subgrade to number of cumulative standard axles to cause a rutting of 20 mm depth using the empirical relation as follows: