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Parametric Analysis of Resilient Modulus Modeling for Recycled Asphalt Pavement in Base Layer

Ehab M. Noureldin and Magdy Abdelrahman

The resilient modulus (M_R) has been found to be the most important parameter in the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) for the base layer. A literature review showed that the use of recycled asphalt pavement (RAP) in the base layer had many economic and structural benefits. The prediction of M_R with the specific MEPDG model for the base layer mixed with RAP was found to be reliable for several field conditions. However, further studies of the MEPDG model are needed for an understanding of the physical meaning of each parameter in the model. The most important factor affecting M_R for the base layer is the state of stresses, especially the confining pressure. This type of state of stresses was found in previous research to be more effective than deviator stresses. The present study focuses on the parametric analysis of each constant in the model versus various confining pressure states under various field conditions and at various RAP concentrations. This parametric analysis is compared with the traditional granular coarse aggregates used for the base layer before RAP is used so that any difference in the physical meaning when RAP is mixed in the base layer blend can be determined. The prediction of M_R by the MEPDG model appears to be sound for RAP, but only with confining pressure levels below 10 psi and without significant variation in water content and maximum dry density. Freeze–thaw cycles do not negatively influence the prediction of M_R for both RAP and granular coarse aggregates.

The resilient modulus (M_R) of unbound layers is a required property during any mechanistic or mechanistic–empirical analysis for flexible pavements. The M_R test is commonly conducted in the laboratory to characterize the stiffness and elasticity responses of the base material (*1*). Proper characterization of the nonlinear, stress-dependent behavior of pavement unbound layers has a significant effect on the accuracy of pavement response predictions (*2*). M_R of unbound granular material is affected by several factors, including state of stress, moisture content, dry density, and freeze–thaw action. Researchers have sought to understand and model the effect of these factors on M_R for various granular materials and to understand the effect of those factors on M_R of recycled asphalt pavement (RAP) and RAP–aggregate blends as a pavement base course layer (*3*).

A literature survey of M_R for base layers found that M_R depends more on confining pressure than deviator stress levels (*1*). Several constitutive M_R models have been studied for the base layer, especially when RAP is used. The *Mechanistic–Empirical Pavement Design Guide* (MEPDG) model was found to be the best suitable prediction model for M_R (*4*). However, more parametric analysis is needed for the constants (K) of the model as related to state of stresses to gain an understanding of the physical meaning of each parameter. Therefore, this study focuses on the parametric analysis for each K parameter in the MEPDG model for various levels of confining pressure at the same field conditions usually studied for RAP.

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PROBLEM STATEMENT

The MEPDG model is the most reliable model and is the best fit for predicting M_R of a granular base layer and when RAP is used as a base layer in various field conditions (*3*). However, it has been found that there is no exact relationship between each K parameter and the studied field conditions individually (*4*). These field conditions include various water contents, decreased dry density, and applied freeze–thaw cycles at various RAP concentrations. Therefore, there is a need to assess the physical meaning of each constant in the model, especially for the state of stresses, which is the most influential parameter for M_R for the base layer. Also, there is a need for a further study of model suitability for traditional base coarse aggregates and RAP–aggregate mixes, depending on the behavior of each constant parameter of the MEPDG model at different states of stress.

SCOPE OF WORK

In this model, multiple regression analysis is achieved for the MEPDG model with the measured M_R values for various measured testing parameters. In the first stage, each K parameter is calculated for five levels of confining pressure (3, 6, 10, 15, and 20 psi) with the Excel Solver tool. Six replicates are considered for each level of confining pressure. Each replicate is assigned a value of deviator stress. This stage of the analysis is an addition to a previous study that did not consider the effect of various states of stress on K regression parameters (*4*). This analysis cannot be conducted at various levels of deviator stress because testing sequences required by the NCHRP 1-28A protocol in M_R testing do not group deviator stress as they do confining pressure levels. This regression is repeated under four field conditions:

1. Percentage of RAP (0%, 50%, 75%, and 100%);
2. Six moisture contents, ranging from OMC − 3% to OMC + 2% with 1% increments, where OMC is optimum moisture content;
3. Two levels of compaction, 100% and 90% maximum dry density (MDD); and
4. Two freeze–thaw cycles.

The MEPDG model follows:

$$M_R = K_1 \cdot P_a \left(\frac{\theta}{P_a}\right)^{K_2} \left(\frac{\tau_{\text{oct}}}{P_a} + 1\right)^{K_3} \tag{1}$$

where

$\theta = \sigma_1 + \sigma_2 + \sigma_3$
$= \sigma_d + 3\sigma_3$
= bulk stress (psi),

$\tau_{\text{oct}} = \frac{1}{3} \cdot \sqrt{\left\{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2\right\}}$
= octahedral shear stress (psi),

M_R = resilient modulus (psi),
K_i = multiple regression constants evaluated from M_R tests,
P_a = atmospheric pressure, 14.7 psi (101.5 kPa),
σ_d = deviator stress (psi), and
σ_3 = confining pressure (psi).

LITERATURE REVIEW

The proper modulus for an unbound base course is necessary for good pavement performance. M_R is a commonly used parameter for defining material stiffness. It is similar to Young's modulus and is based on the recoverable axial strain (ε_r) under an imposed axial (deviator) stress (σ_d):

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2}$$

The *Guide for Mechanistic–Empirical Design of New and Rehabilitated Pavement Structures* (*5*) and the 1993 *AASHTO Guide for Design of Pavement Structures* (*6*) recommended the use of the resilient modulus of base materials as a material property for characterizing pavements during structural analysis and design (*7*).

In the laboratory M_R test, constant confining pressure is maintained within a conventional triaxial cell, and a cyclic axial stress is applied to simulate traffic loading. The NCHRP 1-28A test protocol used in this study consists of 30 loading sequences, but the protocol loading involves a conditioning stage, which attempts to establish steady state or resilient behavior, through the application of 1,000 cycles of 30 psi deviator stresses at 15 psi confining pressure. The cycles are then repeated 100 times for 30 loading sequences with various combinations of deviator stress and confining pressure. M_R is calculated as the mean of the last five cycles of each sequence from the recoverable axial strain and cyclic axial stress (*8*).

The M_R prediction model is called mechanistic–empirical because of the mechanistic calculation of stresses, strains, and deflections of a pavement structure. These properties are the fundamental pavement responses under repeated traffic loadings. The relationship of these responses to field distresses and performance is determined with existing empirical relationships, widely known as transfer functions. The design process is an iterative procedure that starts with a trial design and ends when predicted distresses meet acceptable limits based on the desired level of statistical reliability (*9*).

M_R of base materials can be estimated with laboratory repeated load triaxial tests. It also can be estimated from empirical correlations with soil properties or nondestructive test results. Several researchers have proposed correlations of M_R with the stress state (*7*).

The influence of stress state on unbound material stiffness has long been recognized in pavement engineering. For coarse-grained granular soils as the base layer, an increase in stiffness with increasing confining stress is usually the dominant effect. Most soils exhibit both effects of increasing stiffness with increasing confinement and decreasing stiffness with increasing shear (*10*). Witczak and Uzan have proposed a universal model that combines both effects into a single equation (*11*):

$$M_R = K_1 \cdot P_a \left(\frac{\theta}{P_a}\right)^{K_2} \left(\frac{\sigma_d}{P_a}\right)^{K_3} \tag{3}$$

where $K_1 > 0$, $K_2 \geq 0$, and $K_3 \leq 0$. A modification of this equation was adopted for the new national pavement design guide being finalized in NCHRP Project 1-37A (*12*):

$$M_R = K_1 \cdot P_a \left(\frac{\theta}{P_a}\right)^{K_2} \left(\frac{\tau_{\text{oct}}}{P_a} + 1\right)^{K_3} \tag{4}$$

where 1 in the τ_{oct} term is used to avoid numerical problems when τ_{oct} approaches zero. The model presented in Equation 4 shows a clear improvement for some materials and tests, but for others model accuracy is diminished. Trial and error in the NCHRP 1-28A project showed that adding K_4 (as in Equation 5) to the previous model increases accuracy generally, but the model remains inconsistent. For some materials improvement is shown, and for some others the model is less accurate. Because the (+1) model did not consistently show improvement in all tests or for all materials, 1 clearly was not always the right value. Hence, in the last model, as presented in Equation 5, a regression constant, K_5, was introduced instead of +1. As expected, the analysis showed that the model exhibits the best overall goodness-of-fit statistics (*10*):

$$M_R = K_1 \cdot P_a \left(\frac{\theta - 3K_4}{P_a}\right)^{K_2} \left(\frac{\tau_{\text{oct}}}{P_a} + K_5\right)^{K_3} \tag{5}$$

M_R is a primary input to any mechanistically oriented pavement design procedure. The pavement design computer code must be adapted to support the specific predictive equation selected for M_R. NCHRP Project 1-28A made no recommendation about the final form for the M_R model. Selection of the final model must be coordinated with the pavement design method or code in which it is to be implemented. NCHRP Project 1-37A, *Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures,* was just getting underway when NCHRP Project 1-28A was being completed.

After a thorough review of the results of Andrei et al. (*10*) and other studies dealing with the prediction of the resilient response of unbound materials, the NCHRP Project 1-37A team selected the model presented in Equation 4 as the recommended M_R model to be implemented and used in the 2002 design guide. This model was deemed the best compromise for accuracy, ease of implementation, and computational stability (for the case of $\tau_{\text{oct}} = 0$). The model given in Equation 4 was implemented in the 2002 design guide software.

EXPERIMENTAL CONSIDERATIONS

This study was conducted on one source of RAP collected by the Minnesota Department of Transportation (DOT) from a trunk highway. Use of one source avoids the problem of variability in RAP from several sources or sites. Also, this RAP was used because it contains all data needed in the study for requested field conditions. This material was mixed with Class 5 base aggregates (Minnesota DOT) at 50%, 75%, and 100% RAP. For sample homogeneity, the maximum particle size should be less than 10% of the mold size. Therefore, all materials greater than 12.5 mm were replaced by others passing the 12.5-mm sieve and retained on the No. 4 (4.75-mm) sieve. This sizing was the only adjustment made for aggregate gradation in this stage of the research (*13*). For this kind of RAP, the OMC is 5.5% and the MDD is 2,124 kg/m^3; for Class 5 the OMC is 6.4% and the MDD is 2,223 kg/m^3 (*1*). For 50% RAP, the OMC is 5.2% and the MDD is 2,188 kg/m^3. For 75% RAP, the OMC is 5.7% and the MDD is 2,161 kg/m^3 (*1*). For both 50% and 75% RAP blends, weights were calculated according to these percentages and the gradation used was that for normal aggregates used for base layers at each sieve size.

The M_R test was conducted immediately after sample compaction. The target sample size was 6 in. in diameter and 12 in. in height. The sample was subjected to 1,000 load cycles for preconditioning, followed by the 30 load sequences specified by the NCHRP 1-28A protocol, Procedure 1A (*14*). The M_R test was conducted inside a triaxial pressure chamber, which maintained the required confining pressure by applying pneumatic pressure with a suitable air compressor. These results of the measured M_R previously had been achieved by the North Dakota State University research team with the cooperation of the Minnesota DOT, which supplied the material samples. The samples were also subjected to two freeze–thaw cycles before testing for determining the M_R values. There was no specification standard for investigating the freeze–thaw effect on M_R. Various methods for evaluating different construction materials were reviewed. This review showed that two cycles are enough to cause most of the detrimental effects for the base layer. More than two cycles could be dangerous in cold climates such as North Dakota, and this could be investigated in future research. Samples were frozen at −12°F for 24 h, then thawed for 24 h at 75°F (*3*). Some samples failed during the M_R test, and no measured data were available for comparison with predicted values from the MEPDG model.

ANALYSIS OF RESULTS

A review of the literature on the M_R MEPDG model showed that $K_1 > 0$, which refers to the Young's modulus of the material. Also, $K_2 > 0$ refers to stress stiffening, and $K_3 < 0$ refers to shear softening (*10*). Most of the collected results satisfy the trends drawn from the literature for the tested confining pressure levels and field conditions, which included water content, dry density, and freeze–thaw cycles at various RAP percentages in the blend. Analysis of each *K* parameter in the model at the various testing conditions is presented next.

Analysis of K_1

The variation of the K_1 parameter is high for variations in confining pressure in the testing conditions. Therefore, the data collected for this parameter is shown in semilog charts in Figures 1, 2, and 3 to represent K_1 under the variations in the confining pressure factor for all testing conditions at different RAP concentrations. Confining pressure levels can be divided into three categories: low (3 and 6 psi), intermediate (10 psi), and high (15 and 20 psi). Under the variation of water content at different levels of confining pressure and different percentages of RAP (Figure 1), K_1 increases with increasing percentage of RAP exceeding the original values before use of RAP (0%). This relationship is obvious at low confining pressure levels (3 and 6 psi) and most water content levels except at OMC − 3%.

At the intermediate confining pressure level (10 psi), K_1 decreases dramatically, approaching zero on granular coarse aggregates before use of RAP (0%). For RAP used at varying water content levels, K_1 also approaches zero, except at 100% RAP and water content levels ranging from OMC − 2% to OMC.

At high confining pressure levels (15 and 20 psi), K_1 increases dramatically at 0% RAP, as it does when RAP is used. However, it does not exceed the original values at 0% RAP. There is an exception for this relationship at low water content levels OMC − 2% and OMC − 3% as K_1 does not increase dramatically like the other water content levels and it decreases with increasing percentage of RAP.

When MDD decreases from 100% to 90% (Figure 2), at low confining pressure, K_1 increases with an increasing percentage of RAP exceeding the original values of K_1 at granular base coarse aggregates (0% RAP). At intermediate confining pressure, K_1 decreases dramatically, approaching zero for all percentages of RAP. At high confining pressure levels, K_1 increases dramatically for all percentages of RAP. In general, K_1 values at 90% MDD are higher than 100% MDD in the other equivalent testing conditions.

For the last case of freeze–thaw cycles (Figure 3) at low confining pressure levels, K_1 increases with an increasing percentage of RAP exceeding the original K_1 values at 0% RAP. This relationship is clearer at the 3-psi confining pressure than at 6 psi. At intermediate confining pressure, K_1 decreases, approaching zero, except at 100% RAP in OMC + 2%. At high confining pressure, K_1 increases dramatically except at a high percentage of RAP and water content concentration. Generally, K_1 values are not affected significantly by freeze–thaw cycles at the same testing field conditions.

Analysis of K_2

For all testing conditions, K_2 is positive at most confining pressure levels and reaches its maximum values for most of the testing conditions at a confining pressure of 10 psi. Therefore, K_2 appears to be more reliable at an intermediate confining pressure than others. Figures 4, 5, and 6 show that K_2 values are very close to zero and are sometimes negative. This result satisfies results from the literature for this model and confirm that K_2 values should be positive for the base layer. This analysis describes the behavior of stress stiffening (*10*).

For water content variation (Figure 4), the maximum values of K_2 are usually reached when water content is far from OMC (at OMC − 3% and OMC + 2%), especially at 50% and 100% of RAP, where K_2 values at OMC are lower than 0% and 75% of RAP. However, this relationship is not the same for granular base aggregates as shown at 0% RAP; the maximum values reached at OMC − 1% and OMC + 1%. The comparison of RAP concentrations and granular base aggregates shows that the trend of K_2 appears to be very close at 75% RAP. Generally, maximum K_2 values were reached in water contents far from OMC and ranged between 50 and 80 at 50% RAP, between 70 and 140 at 75% RAP, and around 50 at 100% RAP. For granular base aggregates, values range between 75 and 100 at water contents close to OMC.

When MDD is decreased to 90% (Figure 5), K_2 values increase, reaching 170 for 50% and 100% RAP and 250 for 0% RAP. These optimum values are reached especially with high water content levels, such as OMC + 2%. K_2 values at OMC − 2% for 50% and 100% RAP significantly decrease and reach nonrealistic negative values for K_2. This behavior is not the same as that for granular base aggregates (0% RAP) at high water content.

After freeze–thaw cycles (Figure 6) were applied at the different water contents, K_2 values ranged between 65 and 75, especially at OMC + 2% for 50% and 75% RAP. However, at 100% RAP, K_2 values decrease to 55 at OMC + 1%. For granular base aggregates

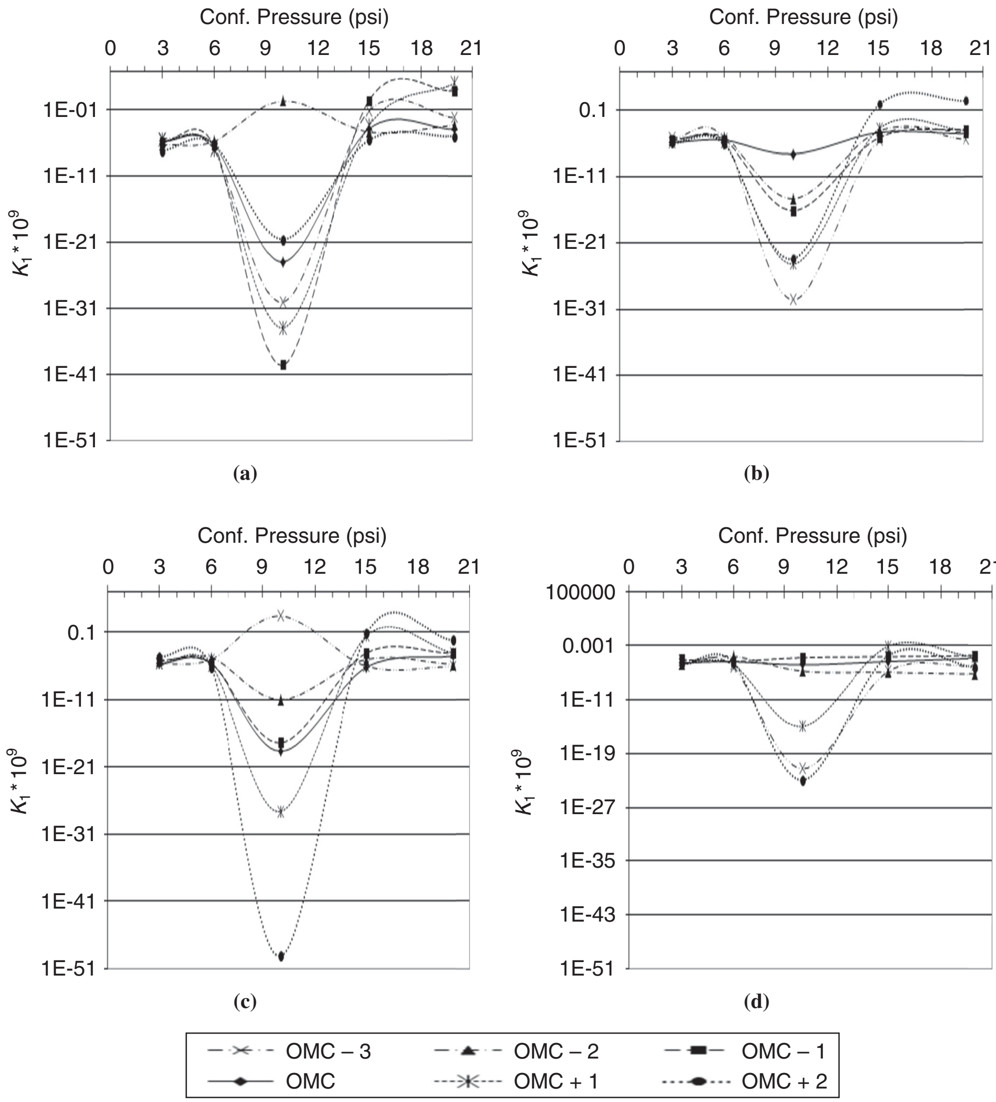

FIGURE 1 K_1 **versus water content variation: water content effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP (conf. = confining).**

(0% RAP), optimum K_2 values reach 80 at OMC + 2% and 65 at OMC. This is much higher than for 50% and 100% RAP. Application of freeze–thaw cycles affects K_2 values positively as it increases after freeze–thaw cycles at the same water content levels for all percentages of RAP.

Analysis of K_3

As shown in Figures 7, 8, and 9 for all testing conditions, K_3 is negative at most confining pressure levels and reaches its minimum values for most of the testing conditions at a confining pressure of 10 psi. This result satisfies those in the literature that state K_3 values should be negative for the base layer. This analysis describes the behavior of shear softening (*10*).

In the case of water content variation (Figure 7), K_3 values reach their minimum values at water content levels away from OMC, such as OMC – 3% and OMC + 2%. This trend is obvious at 50%, 75%, and 100% of RAP. The lowest K_3 values are approximately –80, –140, and –60 for cases of 50%, 75%, and 100% RAP, respectively. At 0% RAP, the lowest K_3 value reaches –110 at a water content level of OMC – 1%. A comparison of 0% and 100% RAP suggests that the general trend of K_3 values at 0% is much lower than 100%, which means that this model is more suitable for granular base aggregates than RAP.

When MDD is decreased to 90% (Figure 8), K_3 values are lower than 100% MDD, reaching –170 at OMC + 2% for 50% and 100% RAP. In the case of 0% RAP, K_3 values are lower than those for all percentages of RAP (50% and 100%) at the three water contents of OMC – 2%, OMC, and OMC + 2%. At 0% RAP, K_3 reaches –250 in the case of OMC + 2%, compared with –170 at RAP 50% and 100%. This result matches the conclusion in the previous case of water content variation.

After freeze–thaw cycles (Figure 9) at the different water contents are applied, K_3 reaches –80 at OMC + 2% for 50% RAP, compared with –60 in the same field conditions before freeze–thaw cycles. K_3 reaches –65 at OMC + 2% for 75% RAP and reaches –50 at 100% RAP and OMC + 1%. For the case of granular coarse aggregates (0% RAP), K_3 reaches –80 at OMC + 2%, compared with –50 at

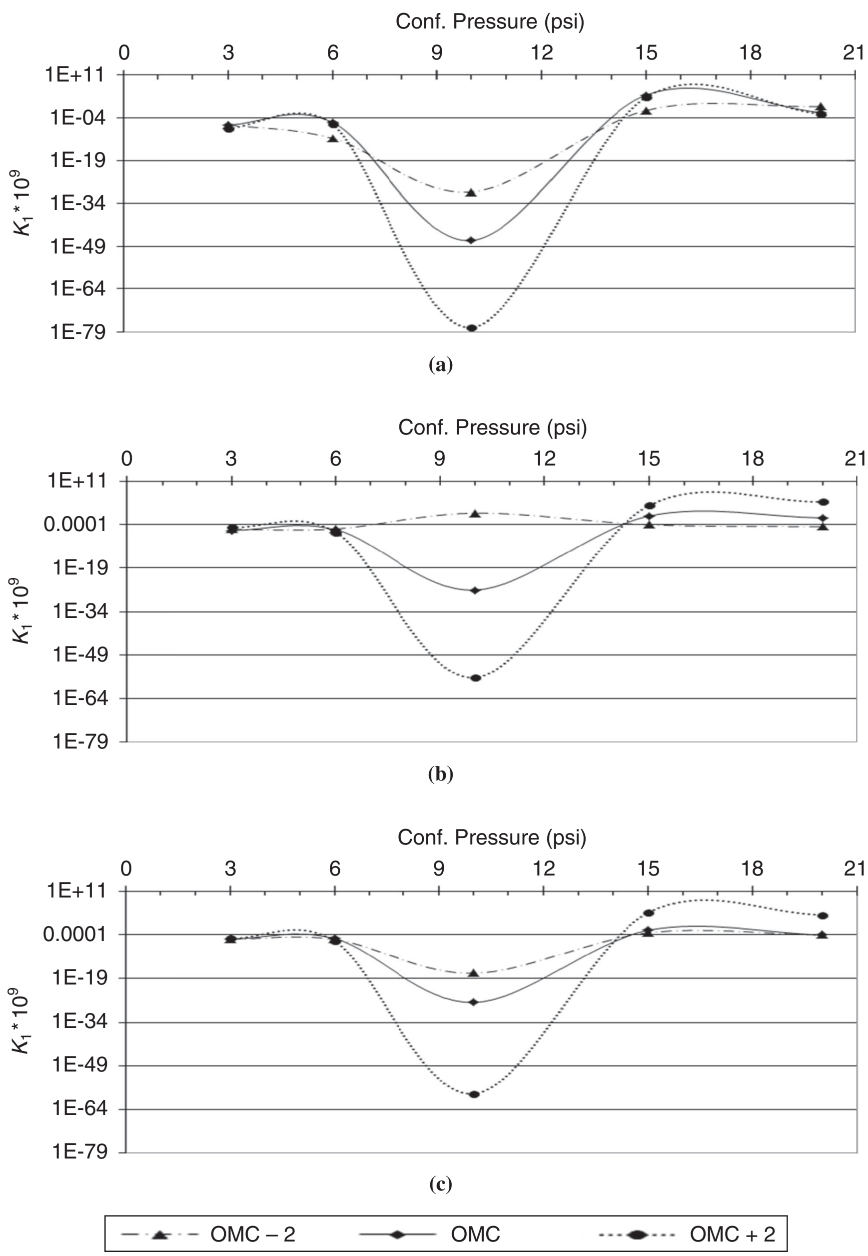

FIGURE 2 K_1 versus MDD variation: 90% MDD effect at (*a*) 0% RAP, (*b*) 50% RAP, and (*c*) 100% RAP.

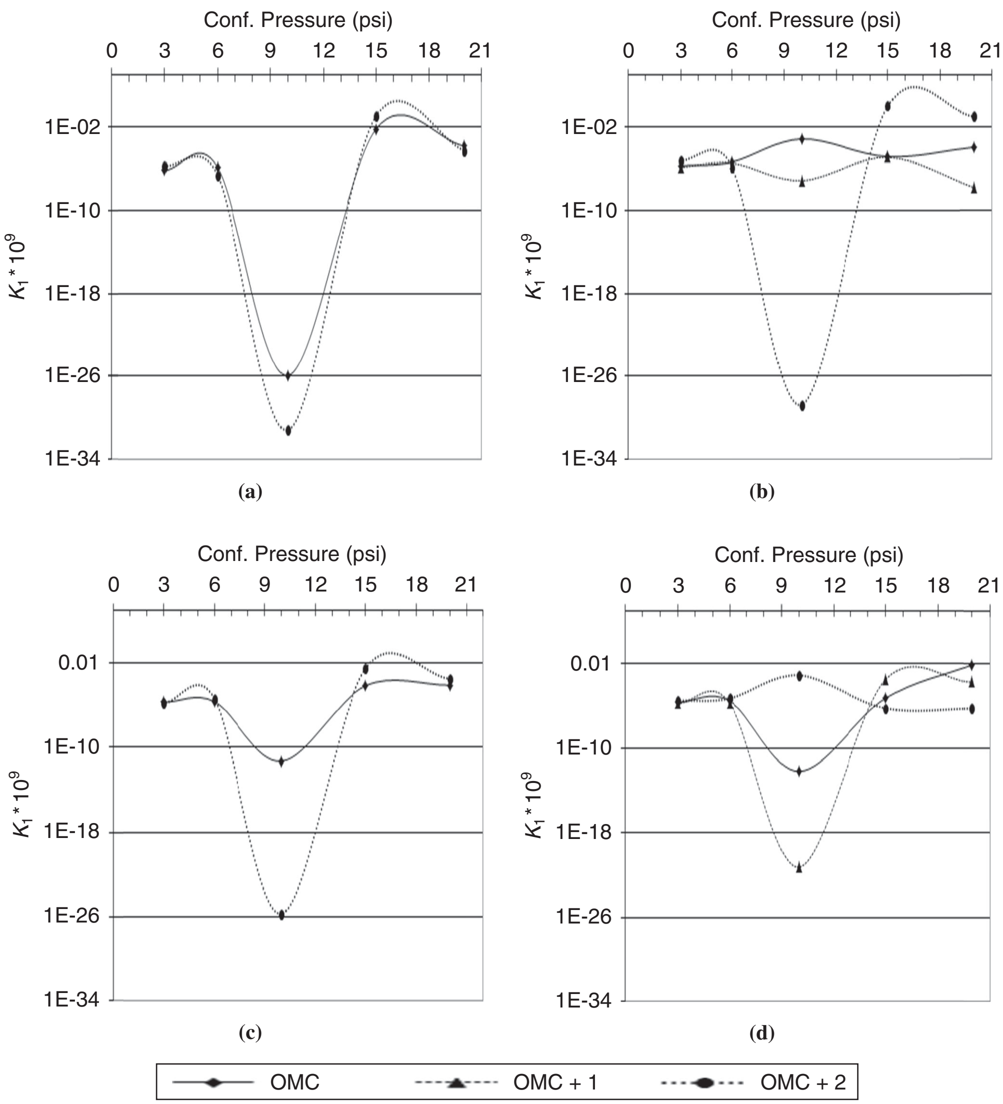

FIGURE 3 K_1 versus freeze–thaw cycles: effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP.

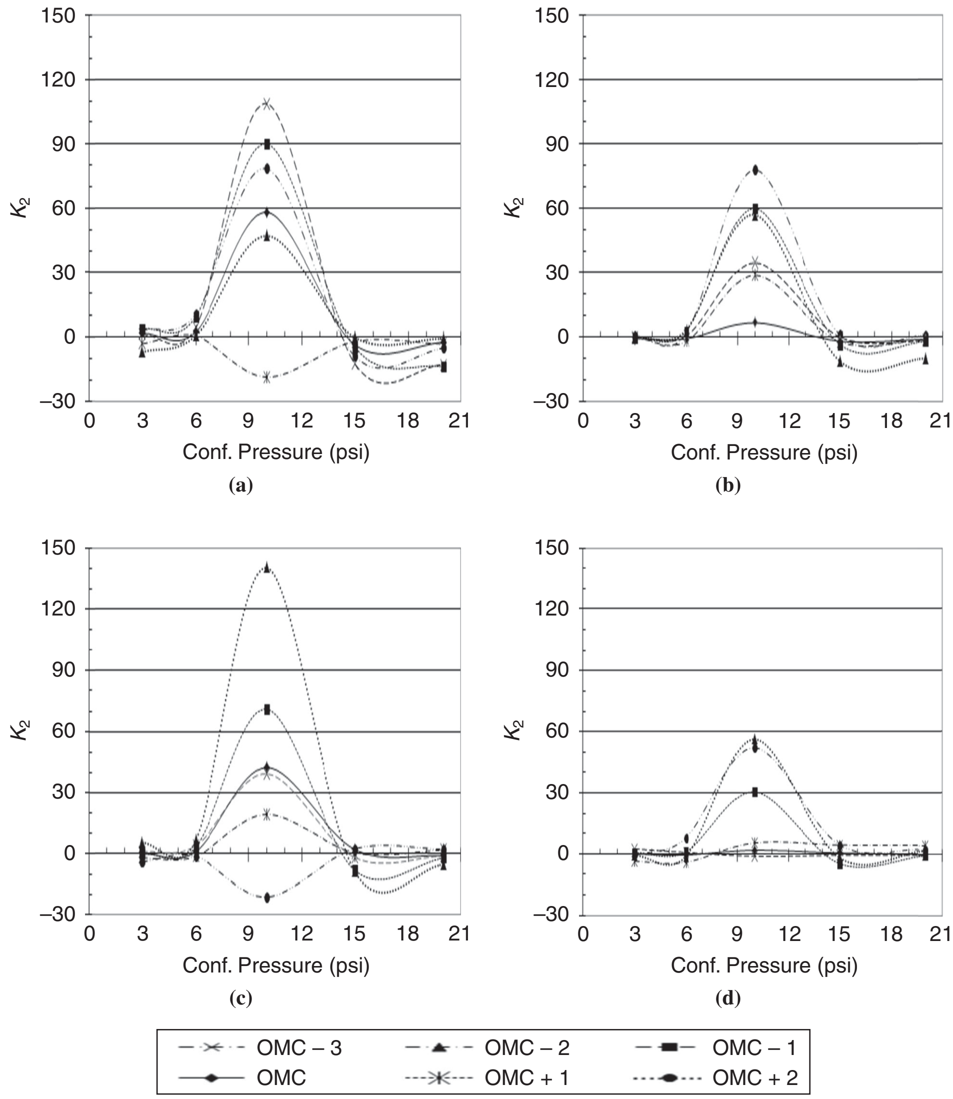

FIGURE 4 K_2 versus water content variation: water content effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP.

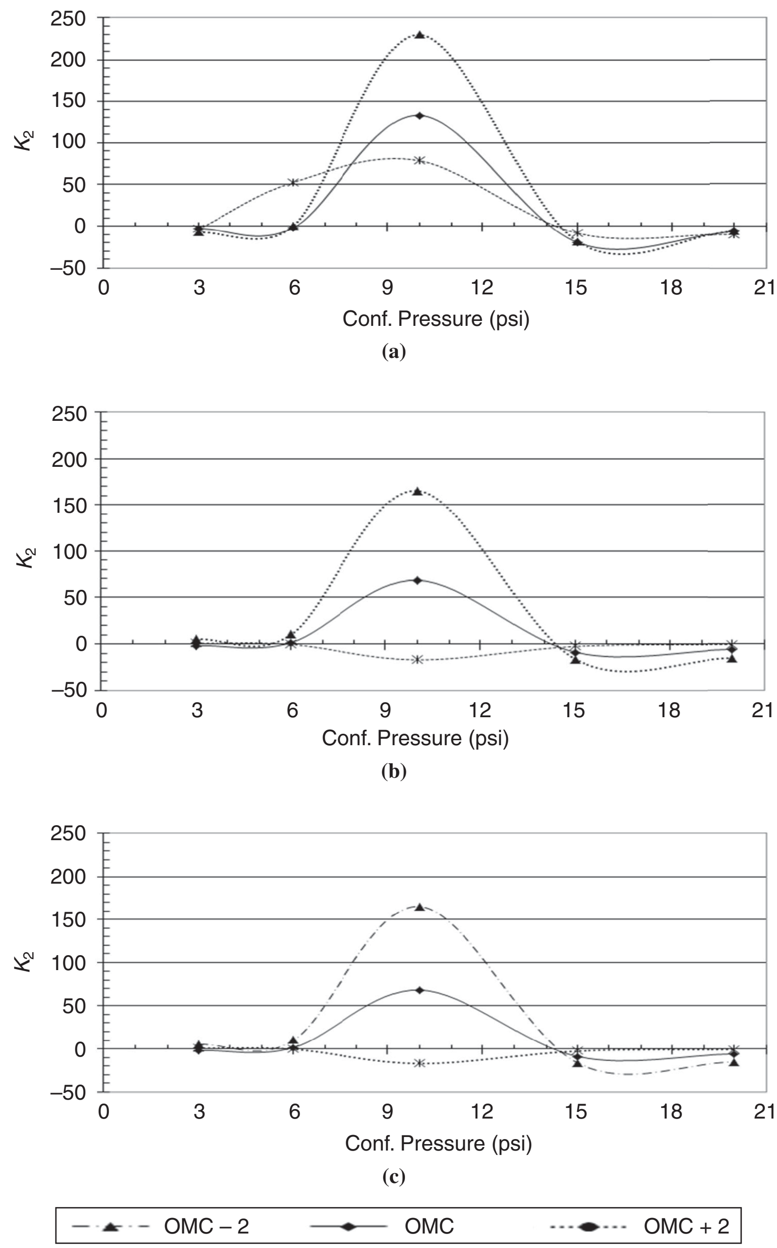

FIGURE 5 K_2 versus MDD variation: 90% MDD effect at (*a*) 0% RAP, (*b*) 50% RAP, and (*c*) 100% RAP.

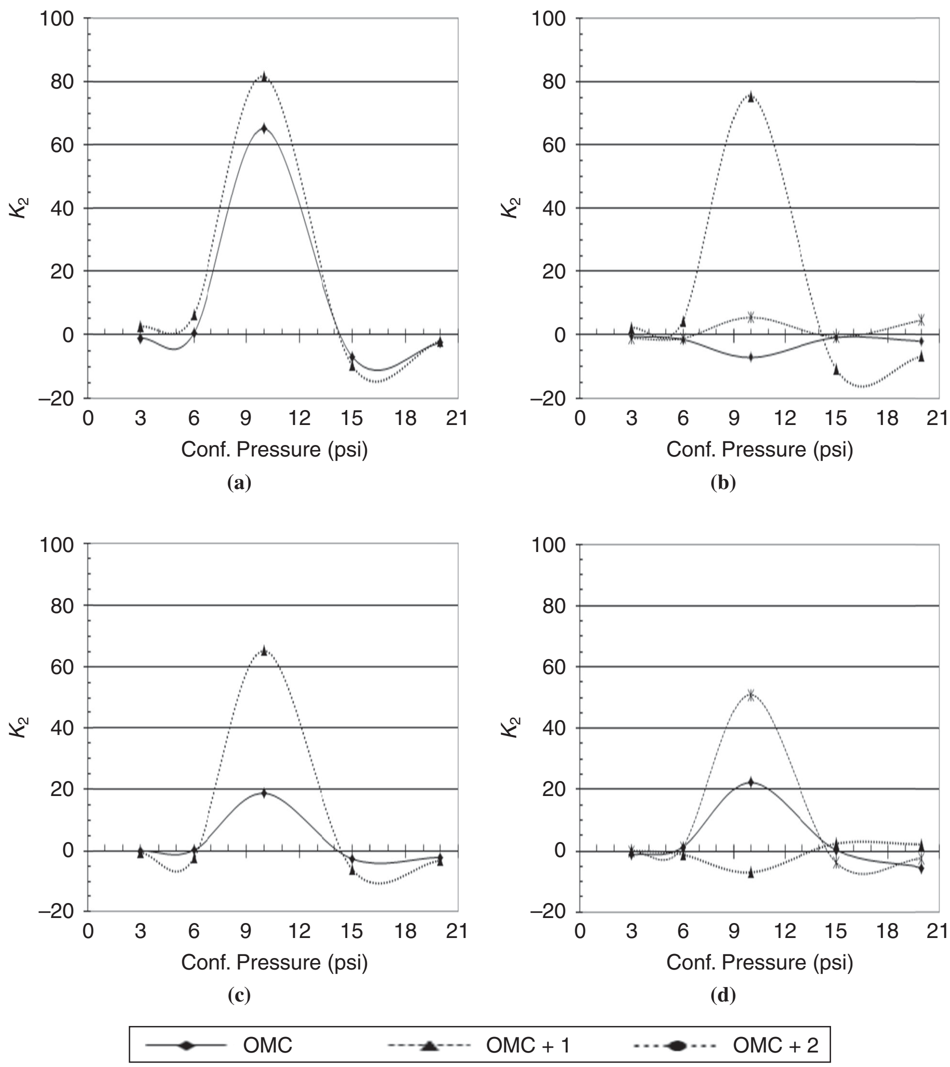

FIGURE 6 K_2 versus freeze–thaw cycles: effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP.

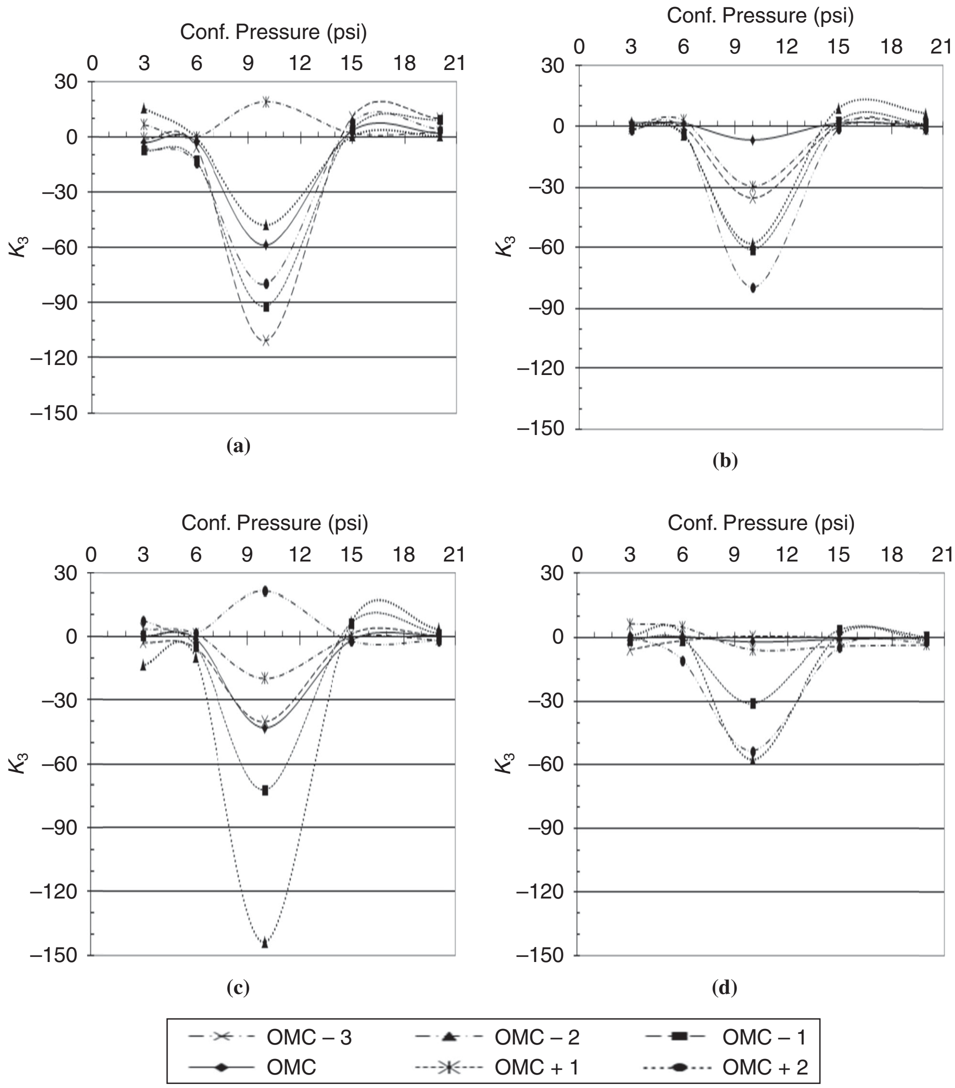

FIGURE 7 K_3 versus water content variation: water content effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP.

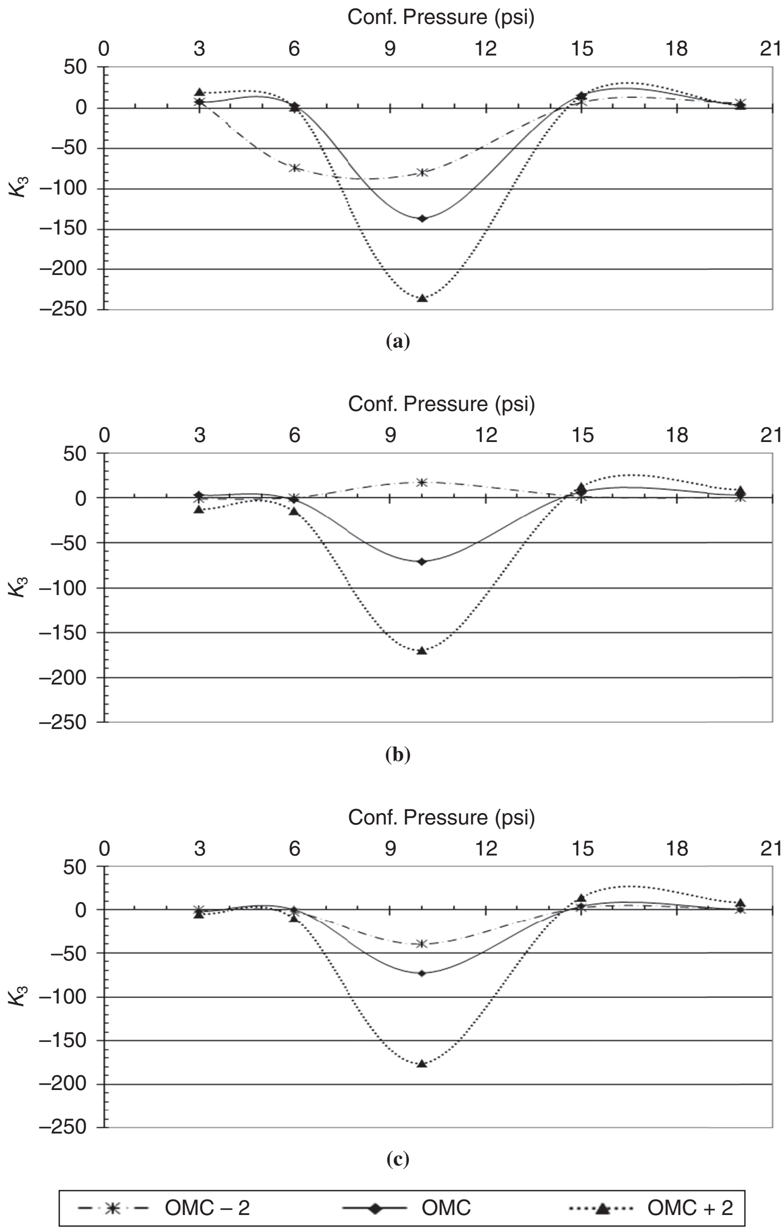

FIGURE 8 K_3 versus MDD variation: 90% MDD effect at (*a*) 0% RAP, (*b*) 50% RAP, and (*c*) 100% RAP.

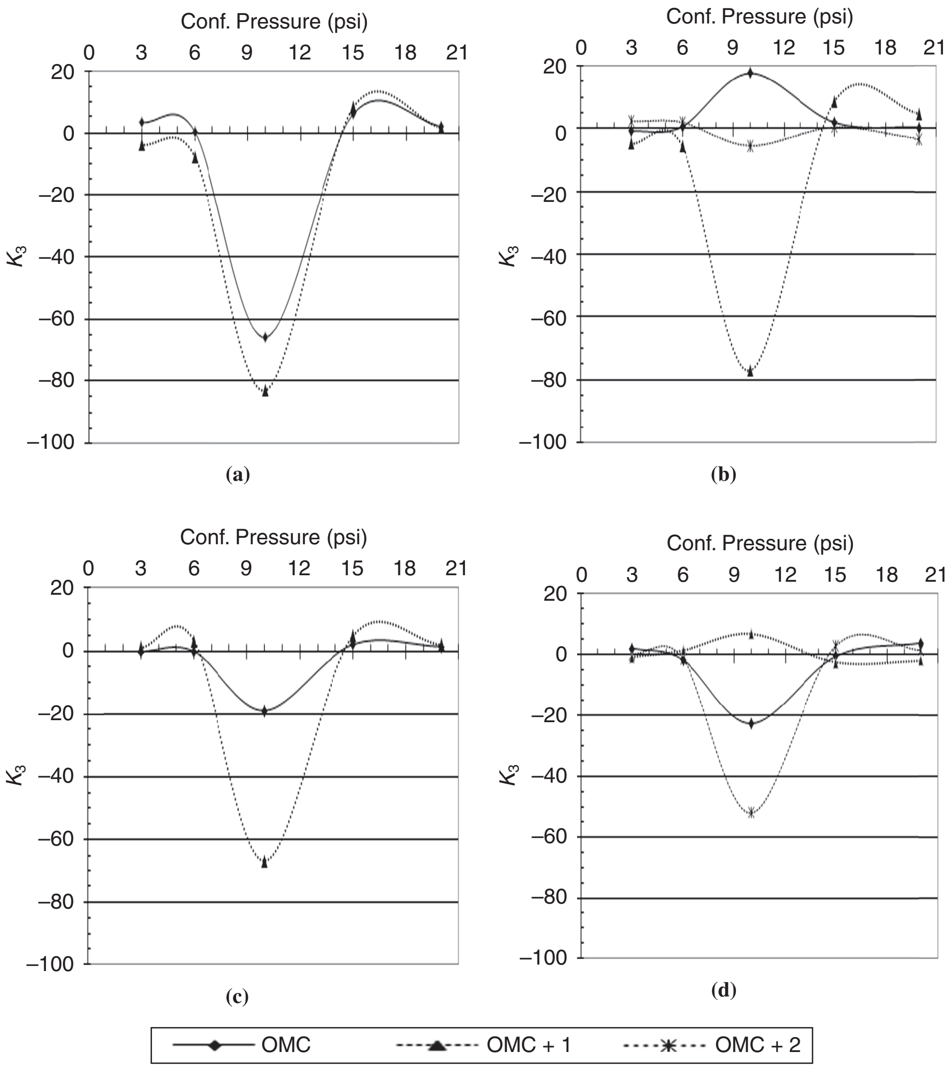

FIGURE 9 K_3 versus freeze–thaw cycles: effect at (*a*) 0% RAP, (*b*) 50% RAP, (*c*) 75% RAP, and (*d*) 100% RAP.

the same case before freeze–thaw cycles. Thus, freeze–thaw cycles affect more positively K_3 absolute values. A comparison of granular coarse aggregates (0%) and RAP (100%) shows that traditional coarse aggregates are more suitable to the model as K_3 absolute values are higher, and this describes shear softening behavior for a base layer better than RAP.

Analysis of RAP–Aggregate Combination

After analysis of each K parameter with various confining pressure levels under various testing conditions, the variation of each K parameter was assessed with different RAP concentrations for all confining pressure levels previously studied. In this case, testing conditions are concerned only with OMC and MDD without freeze–thaw cycles because these are the most frequent field conditions likely to happen early in the life of a pavement section. This stage of analysis is achieved to differentiate between the behaviors of each K parameter when granular base coarse aggregates or RAP is used.

The following analyses are shown in Figure 10. K_1 values are low for granular base coarse aggregates and RAP concentrations (50%, 75%, and 100%) at low and intermediate confining pressure levels (Figure 10*a*). However, K_1 increases dramatically at high confining pressure levels (15 and 20 psi), especially for base coarse aggregates. It increases significantly with increasing RAP concentration.

K_2 values are almost zero for all RAP concentrations (Figure 10*b*) at both low and high confining pressure levels, except for the intermediate level (10 psi). It has high positive value at granular coarse aggregates (0% RAP) compared with 50%, 75%, and 100%. The 75% RAP K_2 values appear to be less sound that those for the other two percentages of RAP (50% and 100%).

K_3 values are almost zero for all RAP concentrations (Figure 10*c*) at both low and high confining pressure levels, except for the intermediate level (10 psi). It has a high negative value at granular coarse aggregates (0% RAP) compared with those of the other percentages of RAP (50%, 75%, and 100%). The 75% RAP values appear to be unreasonable compared with those of 50% and 100% RAP.

The behavior of this model is approximately the same for both granular base coarse aggregates and RAP when used in a base layer. However, if the confining pressure levels are increased to 10, 15, and 20 psi, the model appears to be a better fit for the case of granular base coarse aggregates. In this case, the values of each K parameter are reasonable and agree better with those shown in the literature for the MEPDG M_R model.

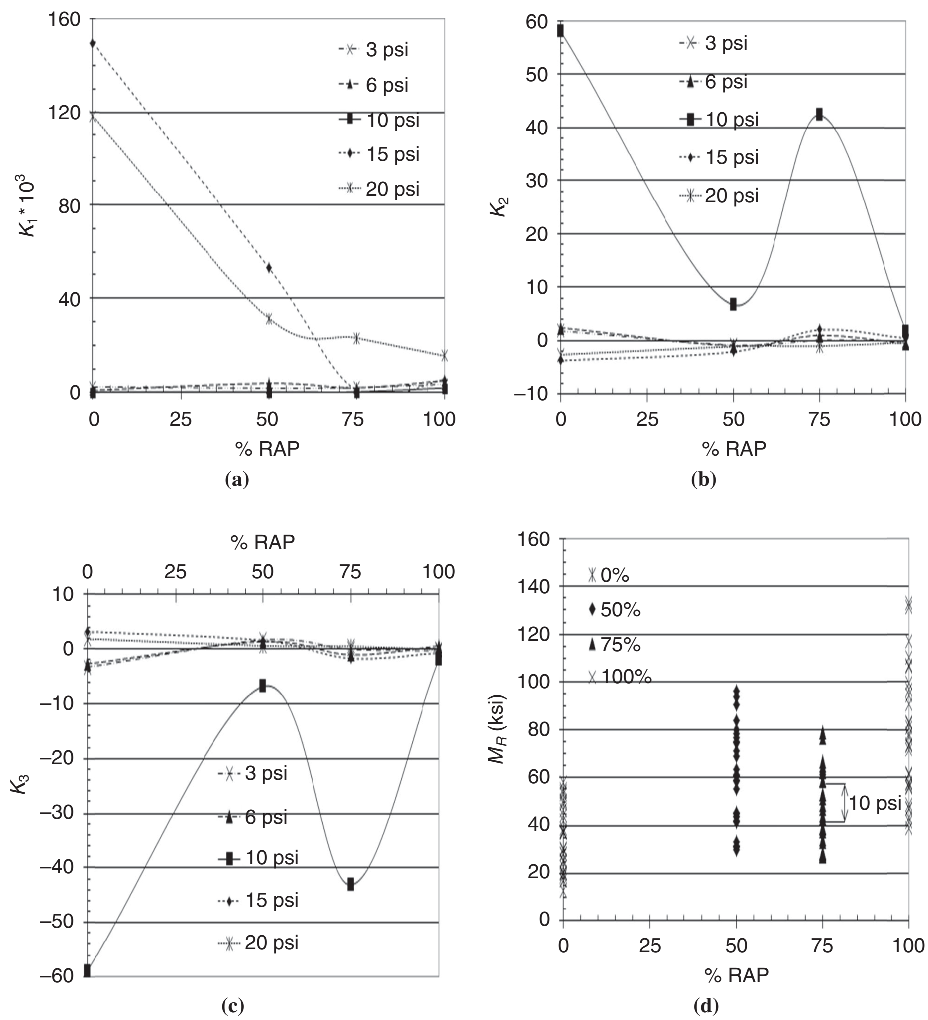

FIGURE 10 Analysis of MEPDG model versus RAP concentration: confining pressure effect on (*a*) K_1, (*b*) K_2, (*c*) K_3, and (*d*) measured M_R versus percentage of RAP.

Proving the variation of K_2 and K_3 at 10 psi for 75% RAP, Figure 10*d* shows that the general trend of the original measured M_R data for 75% RAP was lower than for both 50% and 100% RAP at all tested confining pressure levels. These results were much lower than expected for 75% RAP as the general trend that M_R increases when the percentage of RAP increases.

CONCLUSIONS

Each unitless K regression parameter in the MEPDG M_R prediction model was compared for confining pressure levels and RAP concentrations at various testing field conditions, such as water content, MDD, and freeze–thaw cycles. The findings follow.

K_1 is dramatically affected by confining pressure levels for both base course layer cases: RAP and granular coarse aggregates. However, this variation is minimized at water contents close to OMC for RAP. Generally, K_1 increases with an increased percentage of RAP at confining pressure levels below 10 psi.

K_1 values at 90% MDD are higher than those at 100% MDD for all the same testing conditions for RAP and granular coarse aggregates. However, they are not significantly affected by freeze–thaw cycles under the same testing conditions.

The general trend of K_2 is that it decreases with an increasing percentage of RAP at low confining pressure levels (<10 psi) and vice versa at high confining pressure (>10 psi). Water content variation is an effective factor for K_2 values for the case of RAP. However, this effect diminishes when OMC is reached.

K_2 values increase at 90% MDD for both RAP and granular coarse aggregates. Freeze–thaw cycles do not negatively affect K_2 for both cases. Generally, K_2 values decrease at 90% MDD and freeze–thaw cycles with an increased percentage of RAP in the base layer.

K_2 values at high confining pressure levels above 10 psi are below zero, which contradicts the concept of stress stiffening related to this parameter, as confirmed by the literature survey.

K_3 values are affected by confining pressure variation, especially at intermediate levels. However, this variation is lowest at water content levels close to OMC for the RAP case.

The K_3 value is negative at low confining pressure levels (<10 psi) and water contents close to OMC. This finding appears reasonable and to satisfy the concept of shear softening related to this parameter, as confirmed in the literature. K_3 values are almost the same for the two cases of with or without RAP in the base course.

A 90% MDD is an effective factor for K_3 values for both cases of granular coarse aggregates and RAP, as the absolute values increased compared to 100% MDD at the same testing conditions. However, freeze–thaw cycles are not effective for the K_3 parameter for both cases of with or without RAP in the base layer.

The M_R prediction by the MEPDG model fits the two studied base course cases, which are the traditional base coarse aggregates and three concentrations of a RAP–aggregate combination (50%, 75%, and 100%) for confining pressure levels below 10 psi.

The case of 75% RAP in the base layer showed some extreme absolute values related to the trends of 50% and 100% RAP. This result is repeated in several other testing conditions.

Further research is needed to validate the MEPDG model for M_R prediction of the RAP base layer through testing of extra parameters related to extreme environmental conditions. Also, more study is needed to relate modeling M_R and permanent deformation for the RAP base layer.

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