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Mechanistic-empirical models for reinforced pavements

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ABSTRACT: Road agencies regard the lack of a robust and non-proprietary design model as the principal reason for their inability to use geosynthetics for reinforcement in paved roadway systems. As many agencies move towards the adoption of mechanistic-empirical models for pavement design and analysis, research is needed to identify methods for inclusion of reinforcement products within such a model. This paper describes two projects providing such models. The first is a completed project resulting in a design model based on a mechanistic-empirical approach to pavement design. The second is a project scheduled for completion in 2003 that will provide a mechanistic-empirical design method compatible with the 2002 Pavement Design Guide currently under development in the USA.

1 INTRODUCTION

In a recent survey of USA state transportation agencies, the lack of a robust and non-proprietary design model was cited as the principal reason for their inability to use geosynthetics for reinforcement in paved roadway systems (Christopher et al. 2001). Road agencies have or are moving toward the adoption of mechanistic-empirical (M-E) methods for pavement design. At the core of these methods lies a numerical response model capable of defining key stress and strain parameters for the pavement system. The response model is mechanistic in the sense that it is dependent on the mechanical properties of the individual pavement system constituents. The stress and strain response parameters are then used in empirical damage models to define long-term performance of the pavement system. In this paper, two approaches for inclusion of reinforcement products within a M-E pavement design framework are identified and investigated.

The first results from a completed project where a M-E model was developed and used to provide predictions of reinforcement benefit for a range of pavement design conditions and geosynthetic properties. Regression equations were developed to relate reinforcement benefit to the parameters varied in the study and constitute the basis for the design model. This approach requires that an unreinforced pavement cross-section be designed by any recognized method. The design model is then used to predict reinforcement benefit, which in turn is used to modify the service life and/or the aggregate thickness of the unreinforced pavement.

The second results from an on-going study sponsored by the Federal Highway Administration (FHWA) scheduled for completion in October 2003. In this project, a M-E model is being developed within the framework of the 2002 Pavement Design Guide (2002 PDG) currently being developed for adoption consideration by AASHTO. The project will result in methods for direct inclusion of reinforcement products within the M-E 2002 PDG. This project differs from the first in that the M-E design method developed will be used directly by pavement designers. In addition, the method will provide a flexible and powerful tool for analysis of a variety of reinforced pavement options.

2 DESIGN MODEL

A M-E modeling approach was used to develop a design model based on results from a parametric study using the M-E model (Perkins 2001). The M-E model consisted of a finite element (FE) response model and empirical damage models, and was calibrated from test sections described by Perkins (1999). The M-E model was used in a parametric study involving over 465 analyses of various pavement cross sections and properties. The damage models were formulated to provide a value for extension of roadway service life, defined by a Traffic Benefit Ratio (*TBR*) for each case analyzed. *TBR* is defined as the ratio of permissible traffic loads for a pavement cross-section containing reinforcement as compared to an identical section without reinforcement. Regression equations were developed between *TBR* and the variables contained in the parametric study.

The finite element response model used an elastic-perfectly plastic model for the asphalt concrete. An elastic-plastic bounding surface plasticity model was used for the base aggregate and subgrade soil layers. An orthotropic (direction dependent) linear-elastic model was used for the geosynthetic. The response model was a full 3-dimensional model established to match the conditions present in test sections (Perkins 1999) for which the M-E model was calibrated against. The 3-D model was also necessary to allow for the inclusion of a direction dependent material model for the geosynthetic.

Identical material properties were used for the asphalt concrete and base aggregate for all analyses performed. The elastic-plastic properties chosen for the asphalt concrete represented a resilient modulus of approximately 2500 MPa, which represents a layer coefficient of 0.4 used in the AASHTO 1993 pavement design guide, and was based on an average value of resilient modulus from tests performed on asphalt concrete cores from test sections. The elastic-plastic properties of the base aggregate were calibrated from triaxial tests performed on material from test sections. This model and these properties were then used to simulate a resilient modulus test. The resilient modulus, M_R, determined from these simulations was plotted against bulk stress, θ. A commonly used expression in pavement engineering (Equation 1) was fit to the data. Best fit values for constants k_1 and k_2 were 4460 and 0.63 when M_R and θ are in units of psi. According to the AASTHO 1993 guide, these values correspond to a typical base aggregate in a damp condition. For a typical value of θ equal to 138 kPa (20 psi), the resilient modulus obtained corresponds to a layer coefficient of 0.14 used in the AASHTO 1993 guide.

$$M_R = k_1 \theta^{k_2} \qquad (1)$$

Six sets of elastic-plastic properties for the subgrade soil were used to represent a subgrade of varying stiffness and strength. The material model and each set of properties were used to simulate a resilient modulus test. Table 1 provides a summary of the resilient modulus determined from these simulations. Equation 2 was used to relate resilient modulus to subgrade *CBR* strength, where M_R must be in units of psi in Equation 2.

$$M_R = 1500\ CBR \tag{2}$$

Table 1. Resilient modulus and *CBR* for subgrade soil.

Case	1	2	3	4	5	6
M_R (MPa)	5.3	10	20	42	84	154
CBR	0.5	1	2	4	8	15

The orthotropic linear-elastic model used for the geosynthetic permitted the specification of four properties controlling the mechanical behavior of this component of the pavement system. These properties correspond to the elastic (tensile) modulus in the two principal directions of the material (i.e. machine and x-machine), the Poisson's ratio between these two directions, and the shear modulus between these two directions. The first two properties can be related to properties determined from existing tension testing methods (i.e. ASTM 4595). Existing testing specifications allowing for the determination of the last two properties do not currently exist. As shown below, these properties have an influence on the mechanical behavior of the geosynthetic modeled by this technique and have a moderate impact on predicted pavement performance.

The parametric study involved the variation of tensile modulus in the strong direction of the geosynthetic and the ratio between the tensile modulus in the weak and strong directions. Given the lack of suitable testing methods for the determination of values for Poisson's ratio and shear modulus, only two extreme values for each parameter were examined. Values of 0 and 0.5 were used for Poisson's ratio. Values of shear modulus were either 0 or values pertaining to an isotropic material.

Membrane elements were used for the geosynthetic. These elements are capable of carrying load in tension but have no resistance to bending or compressive stresses. The nodes on the geosynthetic were equivalenced with those of the surrounding base and subgrade. Initially, contact interface surfaces between the aggregate and the geosynthetic and between the subgrade and the geosynthetic were used. Misleading results were possible, however, for the types of response measures extracted from the model. In addition, no suitable method currently exists for the determination of small displacement shear interface stiffness, as is needed in this modeling application. The effect of varying interface properties is therefore accounted for empirically by comparison of model predictions to test section results.

In addition to the variation of subgrade and geosynthetic properties, the thickness of the asphalt concrete and base aggregate layers were varied. Variation of these parameters resulted in the analysis of over 465 cases. For each reinforced case, an identical unreinforced pavement was analyzed. For each analysis, the vertical compressive strain in the top of the subgrade and a weighted average of bulk stress for a representative volume of the base aggregate was extracted as response parameters.

Empirical damage models were formulated to use the two response parameters described above. The first damage model is known as a subgrade rutting criterion and is given by Equation 3

$$N_{12.5mm} = A\,\varepsilon_v^{\,-B} \tag{3}$$

where $N_{12.5mm}$ is the number of traffic loads necessary to reach 12.5 mm of pavement deformation, ε_v is the vertical compressive strain in the top of the subgrade, and *A* and *B* are material parameters (Huang 1993). Parameters *A* and *B* were calibrated from test sections and were found to be $1.8\text{x}10^{-5}$ and 4.07, respectively. The ratio of traffic loads given by Equation 3 for equivalent reinforced (*R*) and unreinforced (*U*) pavement sections provides a direct definition of *TBR* (Equation 4). For this damage model, reinforcement results from effects in the subgrade, meaning this *TBR* is viewed as a partial *TBR* for subgrade effects (TBR_S).

$$TBR_S = \left(\frac{\varepsilon_{v-R}}{\varepsilon_{v-U}}\right)^{-B} \tag{4}$$

The bulk stress response parameter was determined for comparative reinforced and unreinforced sections. The bulk stress was used in Equation 1 to calculate a base aggregate resilient modulus for each case. These values were then used in the AASHTO 1993 flexible pavement design equation, with all other parameters being equal, to determine the ratio of traffic loads between the reinforced and unreinforced sections. This *TBR* was viewed as a partial *TBR* for reinforcement effects in the base layer and is denoted as TBR_B. Total *TBR* for the reinforced pavement is then computed from Equation 5, whose validity can be established within the context of the AASHTO 1993 method.

$$TBR_T = TBR_S \times TBR_B \tag{5}$$

These values of *TBR* correspond to no reduction in base aggregate thickness and are denoted as $TBR_{BCR=0}$, where *BCR* denotes Base Course reduction Ratio and is defined below.

Computed values of TBR_S and TBR_B from the M-E model were related to the parametric study variables through a series of regression equations, which form the basis of the design model. The design model was calibrated from test section results described by Perkins (1999). This involved the empirical adjustment of the geosynthetic elastic tensile modulus used in the response model from reported values of wide-width tensile modulus measured at 2 % axial strain, where the latter is used as an input parameter to the model. Similarly, an empirical reduction factor for interface shear resistance was developed by comparison of design model predictions to test section results. This resulted in a reduction factor of 1.0 for the geogrids used in this study and 0.765 for the geotextile used.

Values of *TBR* from the design model were then used to compute a Base Course reduction Ratio (*BCR*) defined as the percentage reduction of base course aggregate for equivalent service life. Computed values of *BCR* for equivalent service life are denoted as $BCR_{TBR=1}$. Combined values of *TBR* and *BCR* are computed based on a selection of either *TBR* or *BCR* between the ranges of $1 \le TBR \le TBR_{BCR=0}$ and $0 \le BCR \le BCR_{TBR=1}$. These selections imply that if values of *TBR* or *BCR* less than the full values are used, then there remains a surplus benefit that can be expressed in terms of the other benefit parameter.

These steps necessary to compute *BCR* were accomplished following the guidelines provided by Berg et al. (2000). This involves the use of *TBR* to define the increased number of traffic loads that can be applied to a reinforced section of the same structural thickness as an unreinforced section. Within the context of the AASHTO 1993 pavement design guide, this is used to define the increased base aggregate structural layer coefficient of the reinforced pavement. This in turn is used in the AASHTO 1993 method to determine a reduced reinforced base thickness having this increased layer coefficient, or some other base thickness yielding a remaining increase in service life. This method was validated by comparison to predictions made from the M-E model where it was observed that conservative estimates of *BCR* and combined *TBR*/*BCR* benefit values are produced.

The design model contains three principal input parameters describing the unreinforced pavement cross-section and five parameters describing the geosynthetic. These parameters are listed along with a brief description in Table 2. Additional details and typical values for these parameters can be found in Perkins (2001). This report also contains information concerning the input of parameters describing the structural and drainage characteristics of the asphalt concrete and base layers and provisions for the inclusion of a subbase layer.

Output from the design model consists of values of TBR_S, TBR_B and TBR_T, and values of BCR for each value of TBR. The design model also computes remaining values of TBR or BCR for specified input values of BCR or TBR, respectively.

Table 2. Design model input parameters.

Parameter	Description
D_1	Asphalt concrete thickness (mm)
D_2	Base aggregate thickness (mm)
CBR	Subgrade CBR (unitless)
$G_{SM-2\%}$	Geosynthetic secant tensile modulus @ 2% strain (kN/m)
G_{MR}	Geosynthetic secant tensile modulus ratio (unitless)
R_S	Reduction factor for geosynthetic interface shear resistance
R_ν	Reduction factor for geosynthetic Poisson's ratio (unitless)
R_G	Reduction factor for geosynthetic shear modulus (unitless)

The design model has been implemented into a spreadsheet-based program (MDT 2001) and was used to generate results shown in Figures 1 and 2. The program contains check boxes for parameters R_ν and R_G. When checked, R_ν corresponds to a Poisson's ratio of 0 and 1.0 when unchecked. For R_G, a checked box corresponds to a shear modulus of 0. Isotropic values are used when unchecked. Figure 1 illustrates the effect of subgrade CBR on values of TBR and BCR where it is seen that reinforcement benefits are more significant for weaker subgrades and become negligible for subgrade CBR values greater than 8. The model also predicts decreasing benefit as asphalt concrete and base aggregate thickness increase. These results are consistent with observations from a variety of test sections (Berg et al. 2000). Figure 2 shows the effect of parameters pertaining to the geosynthetic. Table 3 provides a list of the geosynthetic parameters used to generate these results. All results were for an asphalt concrete thickness of 75 mm, a base thickness of 300 mm and a subgrade CBR of 1.5.

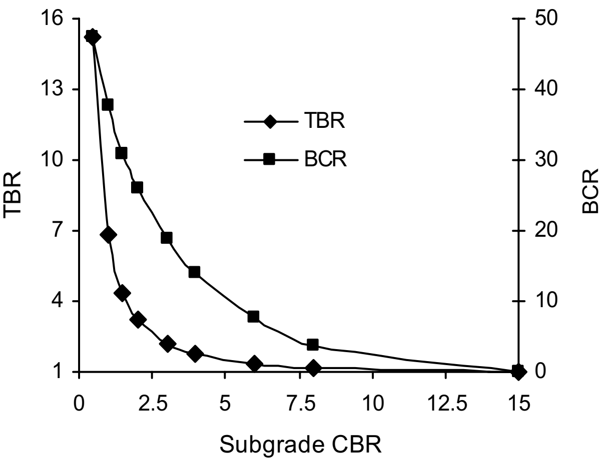

Figure 1. TBR and BCR vs. subgrade CBR.

Table 3. Geosynthetic properties.

Curve	$G_{SM-2\%}$	G_{MR}	R_S	$R\nu$	R_G
Isotropic	Variable	1.0	1.0	unchecked	unchecked
R_G	Variable	1.0	1.0	unchecked	checked
$R\nu$	Variable	1.0	1.0	checked	unchecked
G_{MR}	Variable	0.3	1.0	unchecked	unchecked
R_S	Variable	1.0	0.7	unchecked	unchecked

3 MECHANISTIC-EMPIRICAL (M-E) MODEL

The authors of this paper and their affiliated institutions are currently cooperating on a project sponsored principally by the US Federal Highway Administration and secondarily by several re-

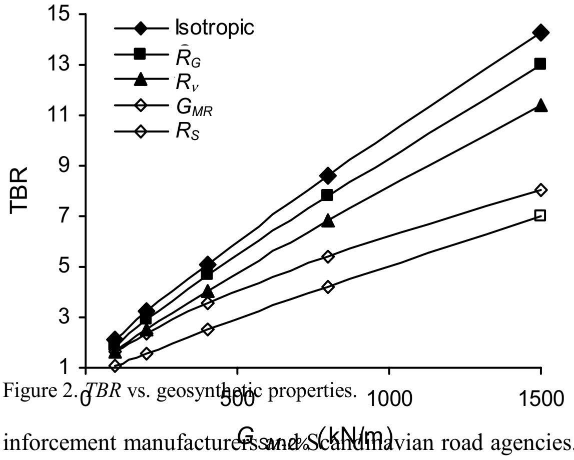

Figure 2. TBR vs. geosynthetic properties.

inforcement manufacturers and Scandinavian road agencies. The principal objective of the project is the development of a design method for reinforced paved roadways that is compatible with the 2002 Pavement Design Guide (2002-PDG) currently being developed for adoption consideration by AASHTO.

The 2002-PDG will be a mechanistic-empirical design method for flexible pavements. The method allows for the use of a finite element response model and empirical damage models to define long-term pavement performance. The response model uses a linear-elastic model including temperature and loading rate effects for the asphalt concrete and non-linear elastic models for the unbound aggregate and subgrade layers. The non-linear model expresses the resilient or elastic modulus of the material, M_R, as a function of bulk stress, θ, and octahedral shear stress, τ_{oct}, and material parameters k_1, k_2 and k_3 (Equation 6). The parameter p_a is taken as atmospheric pressure.

$$M_R = p_a\, k_1 \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{6}$$

The damage model used for definition of pavement permanent surface deformation expresses the accumulated permanent vertical strain, ε_p, at a point in the pavement layer as a function of the vertical resilient strain, ε_r, determined from the response model, the number of load cycles, N, material parameters A and B, and a field calibration factor, β (Equation 7).

$$\varepsilon_p = \varepsilon_r\, \beta\, A\, N^B \tag{7}$$

The damage model calculations in the 2002-PDG explicitly include seasonal variations of material properties (e.g., temperature influences on asphalt concrete stiffness, freeze/thaw variations in unbound material properties). Traffic loading is modeled using a traffic load spectrum in which the traffic mix and volume may also vary over time. Field calibration of the 2002-PDG damage models is currently underway.

Several features of the 2002-PDG are being examined to determine how reinforcement products should be included as structural components in the design procedure. In addition to the model described by Equation 6, four other material models for the unbound pavement layers are being examined to determine the level of material model complexity needed to illustrate reinforcement effects. The first model is similar to Equation 6 but includes anisotropy such that a horizontal modulus of the base aggregate layer that is less than the vertical modulus can be specified. This feature not only has the potential to more accurately model pavement response (Adu-Osei et al. 2001) but may allow for a greater contribution from the reinforcement.

A non-linear isotropic hyperelastic model (Hoff et al. 1998) that includes the recoverable dilatancy during each load cycle is being examined. The model has three parameters; Young's modulus, Poisson's ratio and a dilatancy factor D. The dilatancy effect is proportional to the product of D and the shear strain. Thus, in agreement with observations from triaxial test results, the dilatancy effect increases at increasing cyclic shear stress.

A Mobilized Friction Model, MFM, (Nordal et al. 1989), which is an isotropic elasto-plastic soil model, is being examined. The model allows for full dilatancy control, and can model any observed stress-strain curve including the continuous development of dilation. Finally, the Boyce (1980) model is being examined. In this model, the bulk and shear moduli are both functions of the mean normal stress and the shear stress and allows for modeling of dilatant behavior.

Various methods are being examined for how the reinforcement and its interaction with the surrounding material should be numerically modeled. It is envisioned that the response model will have the added feature of a layer of membrane elements representing the reinforcement with a contact interface on both sides of the membrane. It is anticipated that the inclusion of a reinforcement layer in the numerical response model will result in lateral restraint of the base aggregate and subgrade material, an increase in bulk stress in the aggregate layer leading to an increased stiffness of this layer, a reduction in octahedral shear stress in the subgrade layer leading to an increased resilient modulus, a reduction of vertical stress on the subgrade and a reduction of vertical resilient strain in the pavement layers. Use of this reduced resilient strain in the damage model given by Equation 7 will result in less permanent deformation for the pavement section for a given number of applied traffic loads.

Large-scale cyclic triaxial tests are being performed on unreinforced and reinforced aggregate specimens to assess the effect of reinforcement on the parameters contained in Equations 6 and 7. It is anticipated that reinforcement has the effect of reducing the development of permanent strain for a given number of load cycles and hence effecting the values of parameters A and B in Equation 7. Reinforcement may also have the effect of increasing the resilient modulus through a shift in the k_1, k_2 and k_3 parameters contained in Equation 6.

Various laboratory tests are being performed on geosynthetics and geosynthetic-aggregate systems to define material properties appropriate for use in this application. These tests include wide-width and uniaxial tension tests under various loading rates and cyclic loading regimes, cyclic pullout tests to define a resilient shear interaction modulus and confined tension tests to examine the dependency of geosynthetic tension modulus on normal stress confinement. These tests have been designed to provide material properties pertinent to small strain / small displacement conditions. The cyclic tension and cyclic pullout tests are designed to provide stress conditioning prior to the measurement of material properties. Stress conditioning is a feature also inherent in resilient modulus testing of pavement and unbound aggregate materials where small strain stiffness parameters are sought.

Results from the design method will be compared to results from previously constructed test sections (Perkins 1999), test sections being constructed under another on-going project and test sections constructed as part of this project. A complete set of laboratory pavement material tests are being performed on the materials used in these test sections to provide the necessary input into response and damage models contained in the 2002-PDG and other material models being used in the project.

4 CONCLUSIONS

The use of mechanistic-empirical pavement design methods represents a significant step in pavement engineering that is being embraced by the USA through the development of the 2002 Pavement Design Guide and in European countries such as Finland, France, Sweden and the UK. Engineers and researchers familiar with geosynthetic reinforcement products have recognized for many years the ability of geosynthetics to provide structural reinforcement to certain pavement configurations. Broad-based acceptance and usage of geosynthetics for this application has been hindered, however, by the lack of robust and non-proprietary design methods. As transportation agencies move toward the adoption of M-E methods, it is imperative that methods be developed for inclusion of reinforcement products within the framework of M-E design methods.

The completed project described in this paper established several essential techniques needed for M-E modeling of geosynthetic reinforced pavements. The outcome of this project was a spreadsheet-based design model that defined reinforcement benefit for a specific unreinforced pavement design. These benefit values are then used to extend the service life or reduce the base aggregate thickness of the unreinforced pavement. With this approach, the design model is not specific to any particular pavement design method.

The on-going project described in this paper will provide a design method or methods compatible with the M-E 2002 Pavement Design Guide currently under development. The methods developed can also be easily adapted to other M-E design methods in use or being considered for use in other countries. This project differs from the first in that the M-E model developed will be used directly as part of the pavement design process.

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