Optimizing Non-linear Granular Layer Coefficients of a Flexible Pavement for Mechanistic-Empirical Method

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ABSTRACT

In recent years, mechanistic-empirical (M-E) design methods are preferred in the design of flexible pavements instead of empirical methods using equations based on the road performance tests. However, the calibration of transfer equations that convert mechanical responses to pavement life and the definition of layer materials used in M-E methods have great importance for M-E methods.

In this study, mechanical analyses of a cross-section designed with the AASHTO-93 method were performed, and service life values were calculated with different empirical transfer equations. The obtained M-E design results were compared with the results calculated with the AASHTO-93 method, and transfer equations compatible with the AASHTO-93 method were determined. Among the transfer equations examined, it was found that the rutting equation of the Asphalt Institute gave the most consistent results with the AASHTO-93 method. In the mechanical analysis of the selected cross-section, granular base and sub-base layers were defined as non-linear elastic reflecting the actual in-situ conditions. K-Ɵ model was preferred for non-linear elastic layer definition, and K_1 and K_2 parameters of this model were optimized.

Keywords: Mechanistic-empirical design, AASHTO-93 method, non-linear layer coefficients, pavement life, rutting, fatigue.

1. INTRODUCTION

Flexible pavements consist of granular base and sub-base layers and a bituminous surface layer built on the subgrade to ensure the comfort and safety of vehicles and to resist the stresses caused by traffic loads and environmental conditions. The main purpose of pavement design is to determine the appropriate thickness of the layers subject to traffic loads and environmental effects. The methods used in flexible pavement design can be examined in five different categories as empirical methods, limiting shear failure method, limiting deflection method, regression method, and mechanistic-empirical (M-E) method [1-4].

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AASHTO-93 method, which uses regression equations related to road performance tests, is preferred by Turkey General Directorate of Highways for the design of flexible pavements [1,5,6]. The AASHTO pavement design guide was first published in 1961 and was revised in 1972, 1981, 1986 and 1993 with the theoretical support added to the knowledge obtained from the road test. The equations used in the AASHTO method were obtained from the Ottowa Illinois road tests and used certain climatic conditions, specific layer materials, and a certain subgrade. The average annual rainfall in the region where the test track is 864 mm and the frost depth is 711 mm. The subgrade has inadequate drainage and the California bearing ratio (CBR) ranges from 2% to 4% [1-3]. In the AASHTO method, the number of load repetitions is a very important point to take into consideration and loads of all types of vehicles in traffic are converted to the equivalent axle load of 8.2 tonnes ($T_{8.2}$) [5]. The present serviceability index (PSI) is located in the design equation based on pavement roughness and distress conditions (fatigue, cracking and patching) [1,4,5]. The low traffic volumes, dated vehicle characteristics, short test times and limited material and climatic properties used in AASHTO road tests have led to questioning the usability of the method. Similar to the AASHTO method, these limitations observed in all empirical methods increased interest in M-E methods [2,4].

Unlike empirical procedures, the M-E design method is more adaptable to changing loading, material, construction, and environmental conditions. In the M-E method, the design is performed with different transfer equations for all types of distress modes (rutting, fatigue cracking, etc.) [2,7,8].

The mechanical part of the M-E design methods is based on basic material mechanics, and stress, strain and displacement values (outputs) are calculated at any depth of the cross-section depending on traffic loads, environmental loads and layer characteristics (inputs) on the road [1,9]. In the empirical part, these mechanical responses are converted to service life by transfer equations (pavement life estimation models) [8].

Burmister's layered theory [1,10] is one of the most practical methods for mechanical analysis of flexible pavements. In this theory, the layers are defined as linear elastic with the modulus of elasticity (E) and Poisson's ratio (υ). The Burmister theory can also be used in mechanical analysis of multilayered systems containing viscoelastic and nonlinear layers with some modifications [1].

Material characterization is a crucial step for mechanical analysis. The mechanical responses of the pavement can be estimated by the theory of elasticity where stresses from external loads are relatively low. However, in real conditions, heterogeneous pavement layers can behave more differently than ideal. Where the bituminous surface layer is thick, the elastic theory applies, whereas in relatively thin surface layers the mechanical response is significantly affected by the nonlinear behavior of the granular layers and the subgrade [11].

The modulus of elasticity calculated based on the recoverable strain of material under repetitive loads is defined as the resilience module (M_R) of the material and is the ratio of deviator stress to recoverable strain in the three-axial test [1]. M_R is the most important mechanical parameter used to characterize the subgrade, sub-base and base layers under repeated loads as the materials used in the layers exhibit permanent deformation under repeated loads [12,13].

In the literature, many studies are showing the variation of flexible pavement performances depending on the M_R values of unbound layers and their characterization;

Shahji (2006) [14] and Masad and Little (2004) [15] stated in their studies that the total rutting on the pavement decreased as M_R increased. Cerni et al. (2012) [16] and Masad and Little (2004) [15] revealed that load-related cracks occur easily due to the decreasing M_R value. In their study, which presented the relationship between M_R and thermal cracks, Şahin et al. (2013) [17] reported that thermal cracks accelerated due to M_R decrease. Masad and Little (2004) [15] reported that the International Roughness Index (IRI) value decreased due to the increase in the base layer M_R.

Numerous models have been proposed in the literature for modeling granular layers and estimating M_R values of unbound granular layers which are highly effective on flexible pavement performance as mentioned above. M_R prediction models are categorized in the literature as follows [13, 18-23];

(1) Empirical regression models

(2) Nonlinear stress-dependent models

(3) Moisture-sensitive models

(4) Moisture-sensitive and stress-dependent models

(5) Stress-dependent and cross-anisotropic models

(6) Moisture-sensitive, stress-dependent, and cross-anisotropic model

(7) Regression models for stress-dependent model coefficients

(8) Regression models for moisture-sensitive and stress-dependent model coefficients

In the studies conducted in the literature, the importance of defining stress-dependency and anisotropic material in the determination of M_R values of unbound layers was emphasized. In the study that investigated the effect of the definition of unbound layers on mechanical analysis, Masad et al. (2005) [24] showed that permanent deformation values increase significantly when non-linear and anisotropic properties are used. Adu-Osei et al. (2001) [25] reported that fatigue cracking life is reduced if cross-anisotropy is used in the base layer. Masad and Little (2004) [15] reported that total rutting and cracking, which led the IRI change, were affected by cross-anisotropy. Tutumluer (1995) [26] denoted that the M_R and Poisson's ratio values of unbound layers are direction-dependent. Hence, cross-anisotropic elastic modeling is a better method to estimate the behavior of these layers than the isotropic elastic model.

In the studies carried out to determine the M_R values of the unbound layers depending on the moisture content [27-31], it was stated that the moisture content is the main parameter especially in the prediction of the subgrade's M_R value, and the high degree of moisture content change decreases the M_R. In the study investigating the effect of moisture content in the base layer on tensile strains under the surface layer, Luo et al. (2017) [23] found that tensile strains increased due to the increase in the moisture content of the base layer.

For the K-Ө, Uzan-Witczak and Lade-Nelson models are among the most well-known models for non-linear granular layer characterization. Among these models, the Uzan-

Witczak model provides the lowest surface displacement and horizontal strain under the surface layer at the same loading conditions, while the Lade-Nelson model gives the highest values. K-Ө model calculated the responses between the results of these two methods [32].

The K-Ө model (Equation 1), proposed by Seed et al. [19] where M_R is a function of the sum of the principal stresses, is frequently used in M_R estimation and is used in the definition of the non-linear elastic granular layer in the current study [33,34].

$$M_R = K_1 \theta^{K_2} \tag{1}$$

In Equation 1, Ө is the sum of the principal stresses in the dynamic three-axial test, and K_1 and K_2 are the regression constants obtained from the experimental data of the material. Because the model is simple and useful, there are many studies in which model parameters are determined for different soil types [35-40].

There are many studies for the use of the M-E method in pavement design, but it is not widely used because of limitations on the calibration of empirical transfer functions [2]. In this study, the empirical transfer equations used in the M-E design method were found to be compatible with the AASHTO-93 for a selected pavement cross-section. Thus, the mechanical responses and pavement life of a cross-section designed with the AASHTO-93 can be determined by the M-E method under different loading, material, and environmental conditions and compared with the AASHTO-93. The present study is original in comparing the transfer equations of the M-E method with an empirical one. Furthermore, the study aims to optimize the nonlinear granular layer coefficients (K_1 and K_2) used in the M-E method, which is highly effective on the mechanical responses and proposed in a wide range in the literature.

As a result of optimizing the layer coefficients, the material types and moisture contents of the unbound layers used in the M-E method will be more specifically defined. Thus, the error rate for defining materials will be minimized in the calculation of cross-section responses.

2. MATERIAL AND METHODS

A conventional flexible pavement cross-section (Figure 1) was selected in the study, and layer thicknesses were determined following the studies in the literature [31,33,41-44]. The mechanical and physical properties of the layers are defined by the Highway Flexible Pavement Design Guide [5] and the maximum number of standard axle loads that the cross-section could be passed, over its economic life was determined by the AASHTO-93 method (Equation 2) [1,5].

$$\log(T_{8.2}) = Z_R x S_0 + 9.36 x \log(SN + 1) - 0.20 + \frac{\log(\frac{\Delta PSI}{4.2-1.5})}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32x \log(M_R) - 8.07 \tag{2}$$

Where,

Z_R: The standard normal deviation of reliability (R) is recommended as -1.645 for highways, -1.037 for state roads, and -0.524 for provincial roads.

S_0: The combined total standard deviation of traffic and performance estimation is suggested between 0.40-0.50 and is used as 0.45.

ΔPSI: The decrease in serviceability index (P_0-P_t). P_0 is the initial serviceability index between 4.2-4.5 when the road is first opened to traffic. In this study, this value was used as 4.2. P_t is the terminal serviceability index and 2.5 for highways and state roads, 2.0 for provincial roads.

M_R: Resilient modulus (psi)

SN: Structural number (inches), is calculated by Equation 3 based on the thicknesses (D_i, D_j) of the layers, the layer coefficients (a_i, a_j) and the drainage coefficients (m_j) of the granular layers.
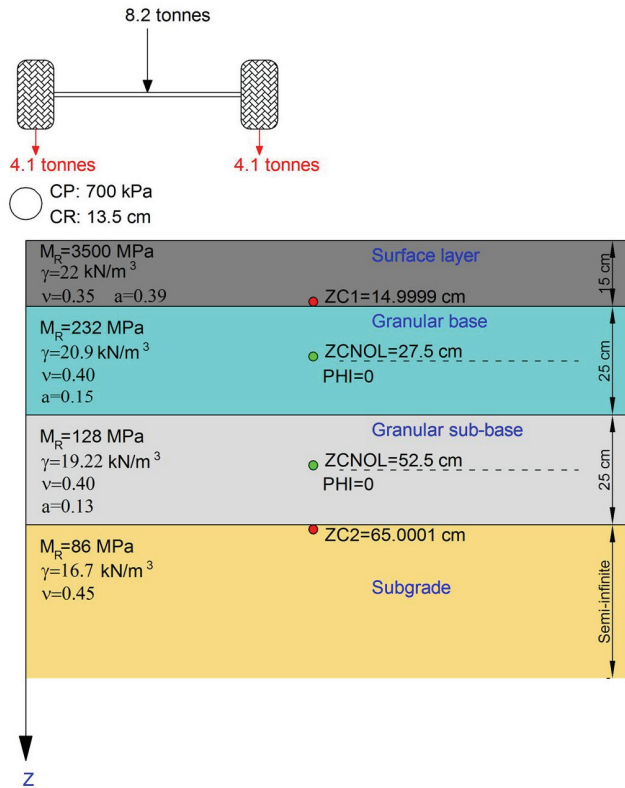

Figure 1 - Model geometry

$$SN = \sum_{i=1}^{k} a_i D_i + \sum_{j=1}^{n} a_j D_j m_j \tag{3}$$

Where,

k denotes the number of hot mix asphalt (HMA) layers and n denotes the number of granular layers. Granular layer drainage coefficients were taken as 1.0. The layer coefficients to be

used in the AASHTO-93 method were obtained with the help of Figure 2, Equations 4 and 5 respectively for surface, base and sub-base layers depending on the layer resilient modulus values and are given in Figure 1 [1].
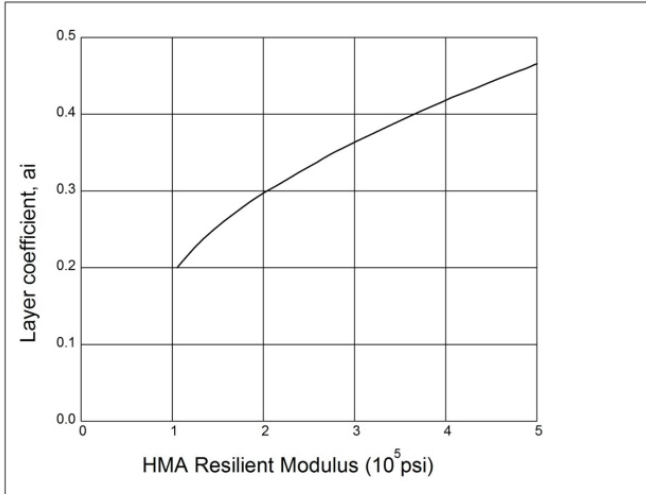

Figure 2 - Layer coefficients due to HMA resilient modulus [1]

$$a_{base} = 0.249x \log(M_{R\,base}) - 0.977 \tag{4}$$

$$a_{sub-base} = 0.227x \log(M_{R\,sub-base}) - 0.839 \tag{5}$$

In Equations 4 and 5, the $M_{R\,base}$ and $M_{R\,sub-base}$ indicate the resilient modulus (psi) of the base and sub-base layers.

In the M-E design method, the mechanical responses (displacement, stress, and strain) should be converted to a parameter that reflects the pavement performance (rutting, fatigue, cracking, etc.). The most commonly used mechanical parameters for this purpose are the horizontal strain (ε_r) under the surface layer (ZC1 in Figure 1) and the vertical strain (ε_v) on the subgrade (ZC2 in Figure 1). If horizontal strains are excessive, cracks occur on the surface, and this is called fatigue criterion. If vertical strains are excessive, deformations occur on the pavement, and this is called the rutting criterion [8,45,46]. In the literature, there are various models for fatigue and rutting criteria, and the general forms of these models are given in Equations 6 and 7, and the coefficients (f1-f5) used in the equations are given in Table 1 [8,47].

$$N_f = f1\varepsilon_r^{-f2}M_{R\,surface}^{-f3} \tag{6}$$

$$N_r = f4(\varepsilon_v)^{-f5} \tag{7}$$

Where N_f is the number of load repetitions required for fatigue, N_r is the number of load repetitions required for rutting distress, $M_{R\ surface}$ is the resilient modulus (psi) of the surface layer.

Table 1 - Distress model coefficients for different agencies [8,47]

	Asphalt Institute	Shell Research	US Army Corps of Engineers	Belgian Road Research Center	Transport and Road Research Laboratory	Federeal Highway Administration	ILLINOIS Department of Transportation	Austin Research Engineers
f1	0.0795	0.0685	497.156	4.92E-14	1.66E-10	0.1001	5.00E-06	4.88E-01
f2	3.291	5.671	5	4.76	4.32	3.565	3	3.0312
f3	0.854	2.363	2.66	0	0	1.474	0	0.6529
f4	1.37E-09	6.15E-07	1.81E-15	3.05E-09	1.13E-06	---	---	---
f5	4.477	4	6.527	4.35	3.75	---	---	---

To determine which of the empirical transfer equations used in the M-E design method yielding consistent results with the AASHTO-93 method, mechanical analysis of the cross-section was performed, and the strain values were obtained under the surface layer and on the subgrade. The obtained values were converted to pavement life and compared with the $T_{8.2}$ values of the AASHTO-93 method, and the transfer equations compatible with the AASHTO-93 method were obtained. KENLAYER software was used for mechanical analysis, which can be used successfully in the determination of displacement, stress and strain values of multilayered systems defined as linear elastic, non-linear elastic and viscoelastic under single, dual, dual tandem and dual tridem loading [1,7,48,49].

The layer thicknesses of the section are defined on the z-axis, and layer parameters (M_R and υ) and unit weights of the layers are given in Figure 1. The wheel load is defined as 4.1 tonnes on the single wheel following the AASHTO-93 method. The contact pressure (CP) of the wheel was selected as 700 kPa [50], and the circular wheel contact radius (CR) that would provide this internal pressure was determined as 13.5 cm. During the mechanical analysis, the contact surfaces between the layers were selected as bonded.

Table 2 - Non-linear layer coefficients

Layer	K_0	K_1 (kPa)	K_2	PHI	ZCNOL(cm)
Base	0.67	Variable	Variable	0	27.5
Sub-base	0.67	Variable	Variable	0	52.5

In addition to the linear elastic model, to perform non-linear analysis and to optimize non-linear layer coefficients, the base and sub-base layers are defined as non-linear using the

coefficients given in Table 2 [51]. To optimize the non-linear layer coefficients in accordance with the AASHTO-93 method, the K_1 and K_2 coefficients of the base and sub-base layers were determined as variables and analyses were made for 60 different K_1-K_2 combinations for the base layer and 360 combinations for the sub-base layer.

In Table 2, K_0 is the soil pressure at rest and is proposed as 0.8 or calculated by Equation 8 according to the Poisson's ratio of the layer [1].

$$K_0 = \upsilon/(1 - \upsilon) \tag{8}$$

K_1 is a non-linear coefficient which varies with the moisture content of the granular layers, and K_2 is a non-linear exponent, the proposed K_1 and K_2 values for granular base layers are given in Tables 3 and 4, and the recommended K_1 and K_2 values for granular sub-base layers are given in Table 5. PHI is the internal friction angle of granular layers. The ZCNOL value is the depth of calculation of the modulus of elasticity for non-linear layers and is recommended to be taken as the mid-depth of the layer [1].

Table 3 - Recommended K_1 and K_2 values for base layers

Reference	Material	K_1 (kPa)	K_2
Hicks (1970)	Partially crushed gravel, crushed rock	11032-34474	0.57-0.73
Hicks and Finn (1970)	Untreated base at San Diego test road	14479-37232	0.61
Allen (1973)	Gravel, crushed stone	12411-55159	0.32-0.70
Kalcheff ve Hicks (1973)	Crushed stone	27580-62053	0.46-0.64
Boyce et al. (1976)	Well-graded crushed limestone	55159	0.67
Monismith and Witczak (1982)	In service base and sub-base materials	19995-53435	0.46-0.65

Table 4 - K_1 and K_2 values based on moisture content for base layers [1]

Mositure content	K_1 (kPa)	K_2
Dry	41369-68948	0.5-0.7
Damp	27580-41369	0.5-0.7
Wet	13790-27580	0.5-0.7

Table 5 - K_1 and K_2 values based on moisture content for sub-base layers [1]

Moisture content	K_1 (kPa)	K_2
Dry	41369-55159	0.4-0.6
Damp	27580-41369	0.4-0.6
Wet	10343-27580	0.4-0.6

3. RESULTS AND DISCUSSION

3.1. Obtaining the Empirical Transfer Model Compatible with AASHTO-93 and Determining the Target Strain

$T_{8.2}$ values for different road types and reliability levels were calculated by the AASHTO-93 method and given in Table 6 depending on the subgrade resilient modulus, layer thickness, layer coefficients, and drainage coefficients given in Figure 1.

Table 6 - AASHTO-93 design results

Road type	R (%)	ZR	Pt	$T_{8.2}(x10^6)$
Highway	95	-1.645	2.5	23.35
State road	85	-1.037	2.5	43.75
Provincial road	70	-0.524	2.0	124.5

As a result of the mechanical analysis of the section by defining the layers linear elastic, the horizontal strain value under the surface layer was obtained as $2.124x10^{-4}$, and the vertical strain value on the subgrade was $2.345x10^{-4}$. Based on these responses, the pavement life values calculated by empirical transfer equations are shown in Table 7.

Table 7 - M-E design results

	Asphalt Institute	Shell Research	US Army Corps of Engineers	Belgian Road Research Center	Transport and Road Research Laboratory	Federeal Highway Administration	ILLINOIS Department of Transportation	Austin Research Engineers
Nf	1303655	1521195.97	765525.96	14952.36	1221238.80	4832.25	521802.87	12472426.51
Nr	24322215	203377885	890750781.7	18800892	46242655.65	---	---	---

Table 7 shows that different transfer equations give quite different pavement life values. In particular, fatigue-related values are very low compared to pavement life due to rutting. When eight values related to fatigue and six values related to rutting are examined, it has been determined that the Nr equations of Asphalt Institute (AI) for highways and Transport and Road Research Laboratory for state roads are 96% and 94.6% compatible with the AASHTO-93 method respectively. For provincial roads with a low level of reliability, a highly compatible model could not be determined.

When the Highway Flexible Pavement Design Guide [5] is examined, the sum of the recommended bituminous layer thicknesses for all R and $T_{8.2}$ values is 19 cm and above (except only R = 70% and $T_{8.2}=3-10x10^6$).

In the current study, the bituminous surface layer thickness was chosen as 15 cm. Therefore, the main failure type for the section was determined as rutting since the surface layer was not thick enough according to the AASHTO-93 method, and therefore the load transmitted to the subgrade increased.

However, the most important point to be noted here is that when the cross-section type and thickness change, the prior failure can be changed, and the fatigue models of the M-E method can also be compatible with AASHTO-93.

Consequently, the rutting equation of AI was chosen as the transfer equation for the optimization of non-linear layer coefficients. In this equation, ε_v value provides the $T_{8.2}$ of the AASHTO-93 method is obtained as the target strain (2.366×10^{-4}) (Equation 7). In the following sections of the study, it is aimed to obtain non-linear layer coefficients which provide this target strain.

3.2. Obtaining Non-Linear Layer Coefficients for Base and Sub-base Layers

The K_1 and K_2 values used in the nonlinear elastic definition of the base and sub-base layers significantly affect the mechanical response values, and these values vary in a wide range according to the type, crush condition, moisture content and granulometry of the aggregate as shown in Table 3-5.
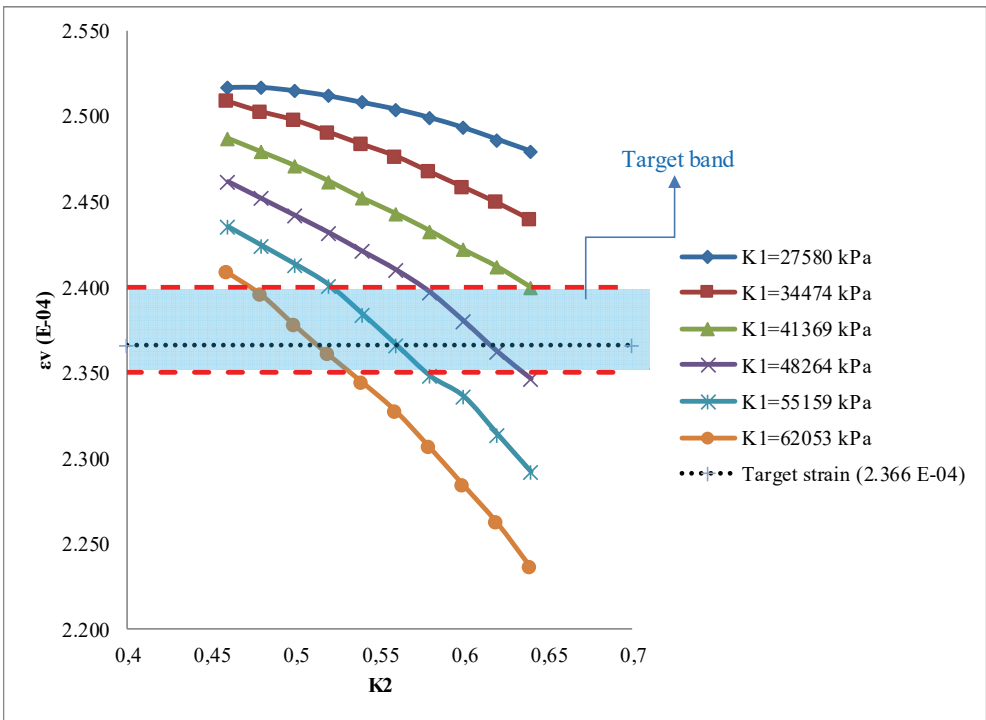

Figure 3 - Analysis results for base layer optimization

If the K_1 and K_2 values can be optimized for base and sub-base layers, a cross-section designed according to AASHTO-93 can be re-analyzed with various loading, material, contact, and environmental conditions and pavement responses and the pavement life values can be determined with M-E design for any distress mode. To determine the non-linear layer coefficients of these two layers, firstly the base layer was defined as non-linear elastic, and the other layers as linear elastic, and K_1 and K_2 values providing the target strain were obtained. The wet moisture content condition is neglected, and K_1 is determined between 27580 and 62053 kPa, K_2 is selected between 0.46 and 0.64 compatible to the literature, so 60 different combinations of K_1 and K_2 were performed mechanically and the results are shown in Figure 3.

In the graph given in Figure 3, K_2 increases for any fixed K_1 value, and K_1 increases for any fixed K_2 value with decreasing ε_v values. That is the deformation resistance of the cross-section increases. The values of K_1 and K_2 of the base layer are not selected only from combinations that provide target strain, instead, combinations from the 2.4×10^{-4}-2.35×10^{-4} band (Figure 3) were selected, and the effects of non-linear definition of the sub-base in the next step were also considered. The target strain line intersects with the curves K_1 = 48264, 55159, and 62053 kPa. No matter how far the lower bound line of target band is positioned from the target strain, an intersection with a new K_1 value cannot be achieved. For this reason, the lower limit of the target band was kept at 2.35 E-04. The upper limit of the target band was determined at 2.4 E-04 to intersect with the curve of 41369 kPa, which is the lower limit value of K_1 for dry aggregate. Thus, the number of values to be worked on for optimization has been increased.

According to Figure 3, the combinations of K_1 and K_2 in the targeted strain band are shown in Table 8.

Table 8 - K_1 and K_2 values that provide the target strain for the base layer

K_1 (kPa)	62053	62053	55159	55159	55159	48264	48264	48264	41369
K_2	0.48	0.52	0.52	0.54	0.56	0.58	0.60	0.62	0.64
ε_v ($\times 10^{-4}$)	2.395	2.36	2.4	2.384	2.366	2.397	2.380	2.362	2.4

With the aim of obtaining the non-linear layer coefficients of the sub-base layer, mechanical analyses with various sub-base parameters were performed for each base combination shown in Table 8, and K_1 and K_2 values were obtained for the target strain. 360 analyses were performed by selecting K_1 values between 27580 and 55159 kPa and K_2 values between 0.4 and 0.6, and the results are given in Figures 4-12.

When the figures giving the results of the sub-base mechanical analysis are viewed, the ε_v values decrease while increasing K_2 at relatively high K_1 values (48264-55159 kPa), and the ε_v values increase as K_2 increases in combinations where K_1 has relatively lower values (27580-34474kPa). According to these results, K_2 value shows a reversible behavior in a weak sub-base layer under a high-strength base layer.
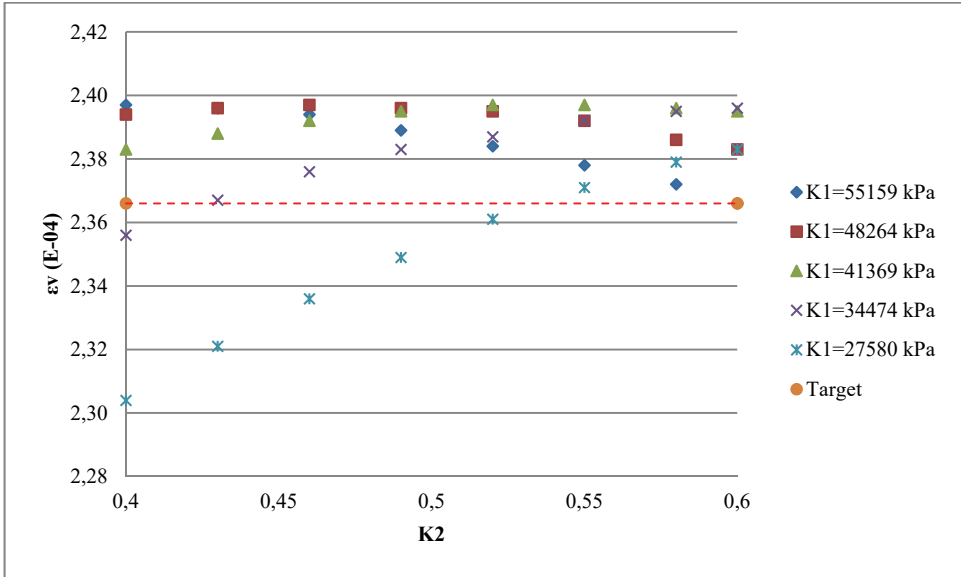
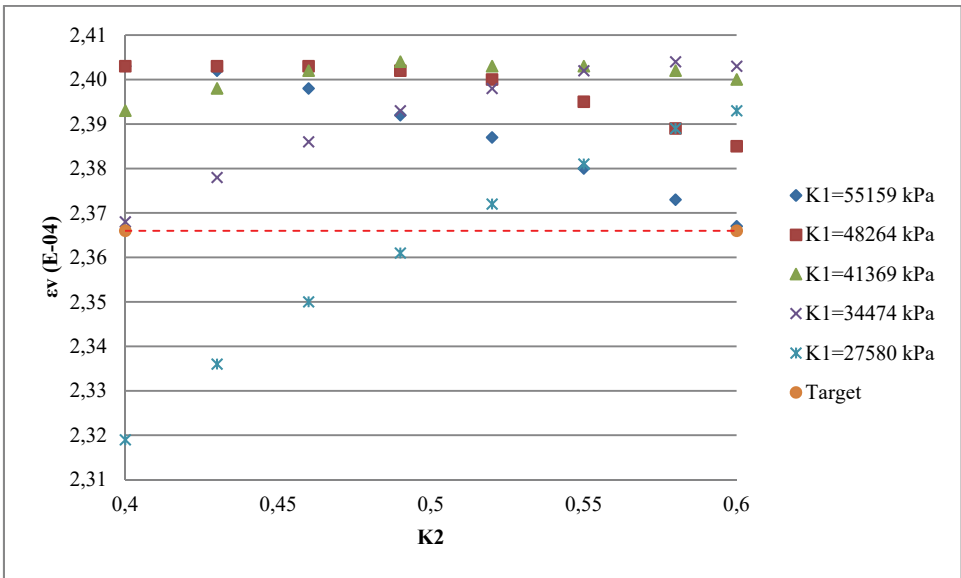*Figure 4 - Analysis results for sub-base layer (Base layer K₁=62053 kPa, K₂=0.48)*

Figure 5 - Analysis results for sub-base layer (Base layer K₁=62053 kPa, K₂=0.52)
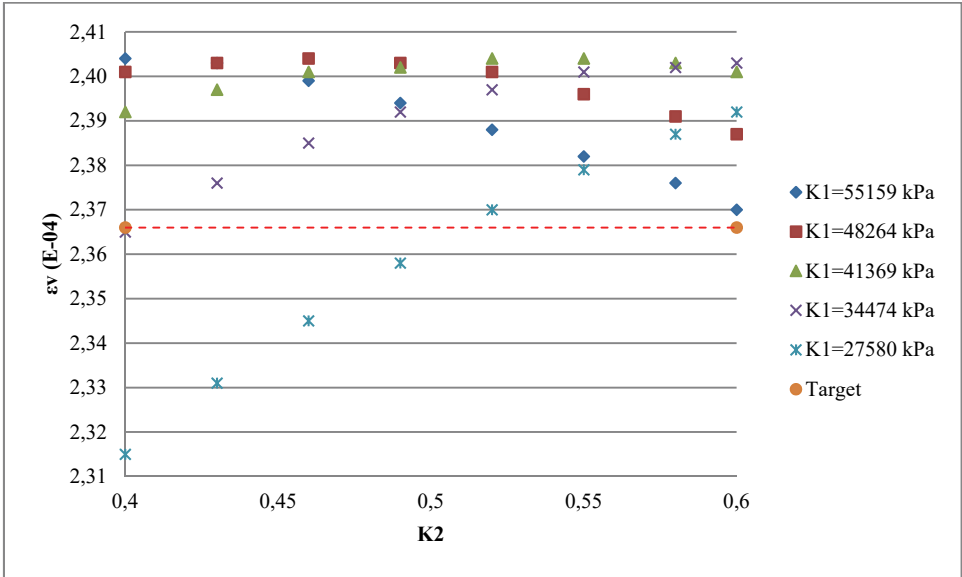
Figure 6 - Analysis results for sub-base layer (Base layer K_1=55159 kPa, K_2=0.52)
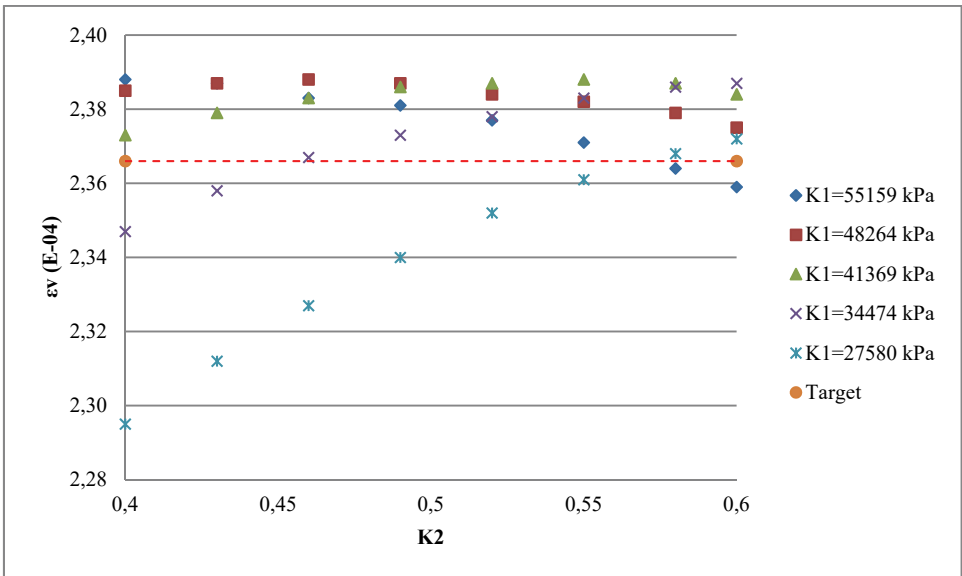

Figure 7 - Analysis results for sub-base layer (Base layer K_1=55159 kPa, K_2=0.54)
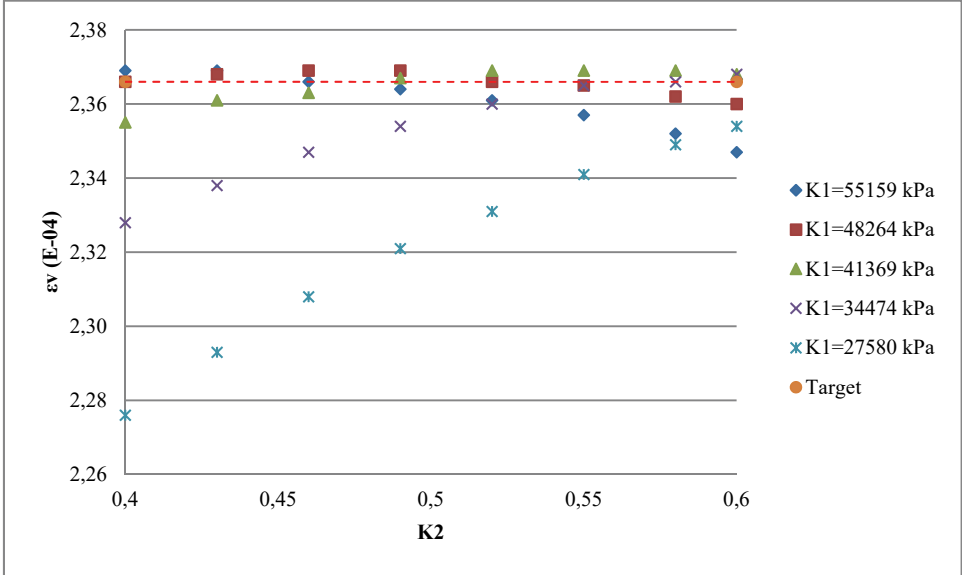
Figure 8 - Analysis results for sub-base layer (Base layer K_1=55159 kPa, K_2=0.56)
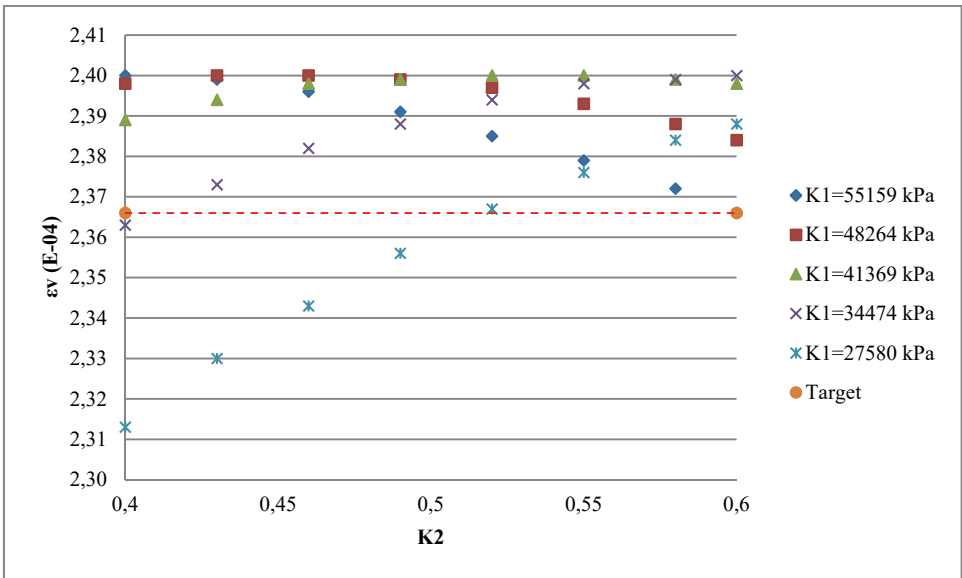

Figure 9 - Analysis results for sub-base layer (Base layer K_1=48264 kPa, K_2=0.58)
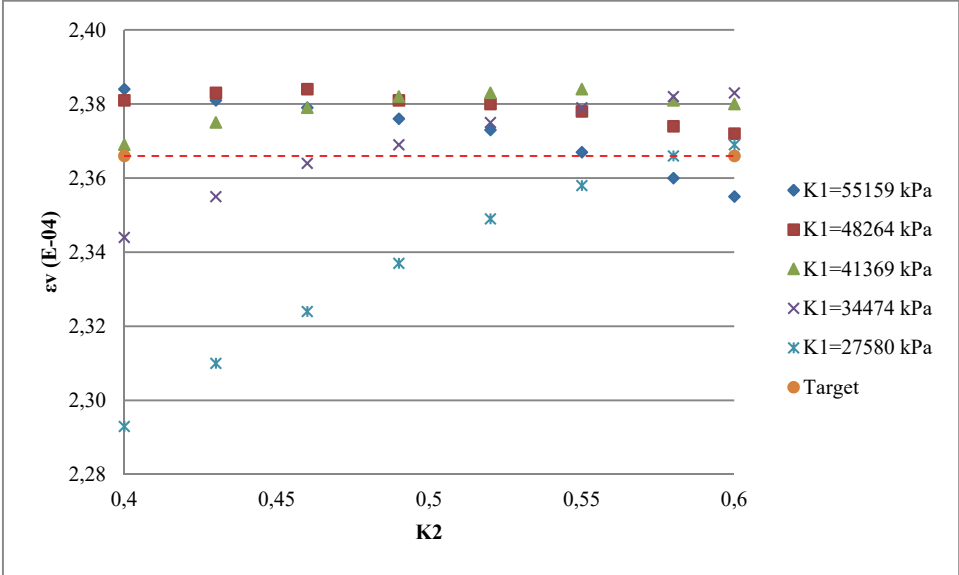
Figure 10 - Analysis results for sub-base layer (Base layer K_1=48264 kPa, K_2=0.60)
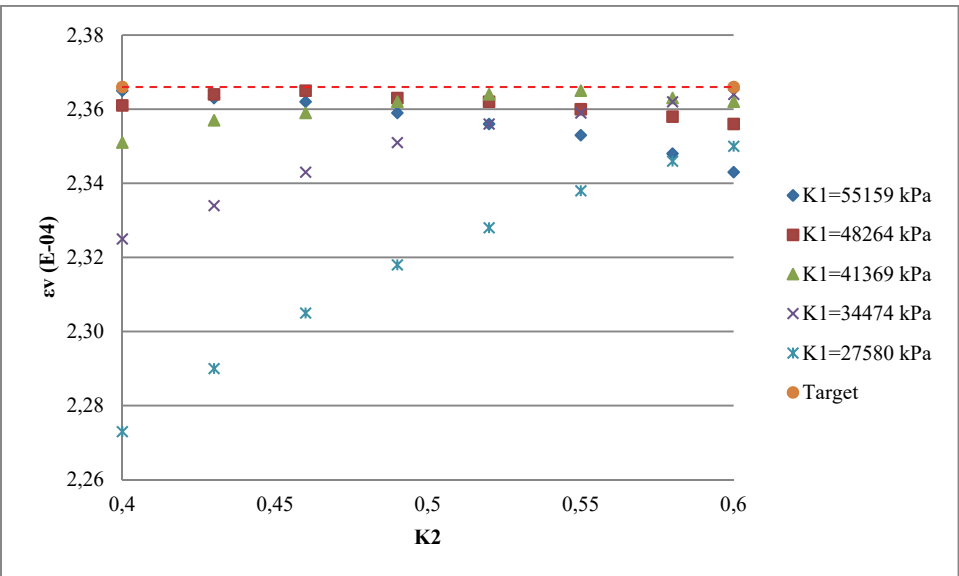

Figure 11 - Analysis results for sub-base layer (Base layer K_1=48264 kPa, K_2=0.62)
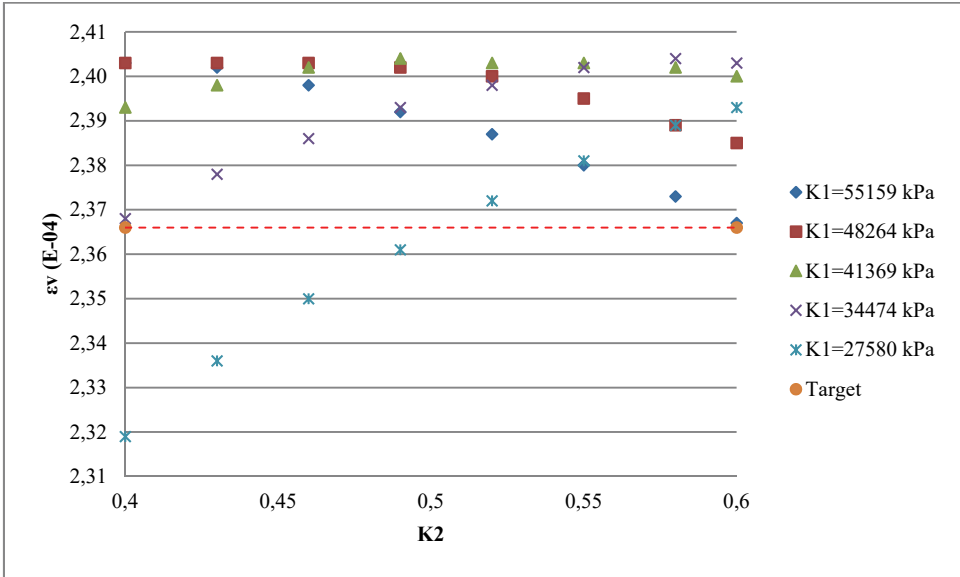
Figure 12 - Analysis results for sub-base layer (Base layer K_1=41369 kPa, K_2=0.64)

In Figures 4-12, the target strain values are shown with dashed lines, and the sub-base K_1 and K_2 combinations that intersect with these lines are determined. Accordingly, the combinations obtained as a result of defining both base and sub-base layers as non-linear elastic are shown in Table 9.

Table 9 - Optimum base and sub-base coefficient combinations

Combination number	Base layer		Sub-base layer		
	K_1 (kPa)	K_2	K_1 (kPa)	K_2	ε_v (x10^{-4})
1	55159	0.56	55159	0.46	2.366
2	55159	0.56	48264	0.40	2.366
3	55159	0.56	48264	0.52	2.366
4	55159	0.56	34474	0.58	2.366
5	62053	0.48	55159	0.60	2.366
6	48264	0.58	55159	0.60	2.366
7	48264	0.60	27580	0.58	2.366

When Table 9 is examined, it is seen that the optimum K_1 values obtained for the base layer are higher than the other tested values and provide high deformation resistance. In the 1, 2, 3 and 4 numbered combinations, the K_1 and K_2 values of the base layer remained the same, while the K_1 and K_2 values of the sub-base layer varied. In particular, when the combinations 2 and 3 were examined, the K_1 values of the sub-base layer under the same base were fixed

as 48264 kPa while the K_2 value increased from 0.40 to 0.52, and this change did not affect the strain values. Accordingly, the importance of the K_2 value of sub-base in pavements consisting of a rigid base and a rigid sub-base can be relatively smaller. Also, when all the base and sub-base combinations in Table 9 are evaluated together, it can be stated that the K_1 and K_2 values used for the M-E design to give similar results with the AASHTO-93 method reflect the dry, crushed and well-graded granular material properties.

According to the Turkish Highway Technical Specifications [52], gravel, crushed gravel, terrace gravel, crushed stone, and crushed slag as a base material, sand, gravel, terrace gravel, decomposed rock, slag, and crushed stone can be used as a sub-base material. When the values obtained in Table 9 are evaluated according to the material properties used in base and sub-base layers, the optimum values obtained for the base layer generally reflect dry and well-graded crushed stone aggregate properties (Table 3). Accordingly, the designer may find that the AASHTO-93 and M-E design methods will yield consistent results for a base layer to be constructed with well-graded and dry crushed stone aggregate. However, when the material type and moisture content used in the base layer change, the M-E design method must be included in the design as well as the AASHTO-93 method. For different K_1 and K_2 values selected from Tables 3 and 4 following the material type and moisture content to be used in construction, the results to be obtained will be different from the results of AASHTO-93.

When the optimum values obtained for the sub-base layer are compared with the K_1 and K_2 values depending on the moisture content given in Table 5, it can be seen that the dry sub-base material is generally compatible with the AASHTO-93 method. However, when using damp sub-base material (combination number 4 and 7), a compatible base layer must be defined.

4. CONCLUSIONS

According to the results obtained from the study, in which the AASHTO-93 method is compared with different empirical transfer models in the M-E design method in terms of pavement life, and the non-linear elastic layer parameters used in mechanical analysis are optimized, the following inferences can be made.

- Axle loads and configurations, tire pressures and patterns, mechanical properties of pavement layers, and climate and environmental conditions are constantly changing. For this reason, in the design of flexible pavements, the M-E design method must be taken into consideration together with empirical methods developed according to certain experimental parameters.

- The empirical pavement life estimation models used in the M-E design proposed by different organizations give quite different results from each other and the AASHTO-93 method. While the mean value obtained from the N_f estimation models is calculated as 2.228×10^6, its standard deviation is 3.908×10^6, and the coefficient of variation is 175%, for rutting models the mean is 236.7×10^6, the standard deviation is 333.99×10^6, and the coefficient of variation is 141%. This high statistical values, especially in the coefficients of variation, show that calibration of the pavement life estimation model in M-E design methods is critical.

- In the flexible pavement design, the main type of deterioration the road will be exposed to should be prioritizedbecause different distress models of the same agency, show highly different design results. When Nr/Nf values are calculated for AI, Shell, US Army Corps of Engineers, Belgian Road Research Center, and Transport and Road Research Laboratory models, the results are obtained as 18.66, 133.7, 1163.6, 1257.4 and 37.9, respectively. When these results are taken into consideration, it is of great importance to estimate the prior distress type of pavement. Considering that the AASHTO-93 method, which is compared with the M-E methods in the present study, yields more consistent results with the transfer equations related to the rutting, the reliability of the AASHTO design decreases significantly for a cross-section subject to fatigue under heavy traffic.

- The cross-section analyzed in this study gave more consistent results with the rutting models in terms of layer thickness (especially 15 cm surface layer). However, if the cross-section type changes and thicker surface layers are used in the surface layer, the prior distress type may return to fatigue. In this case, while the unbound layer coefficients are optimized, tensile strain under the surface layer should be used as the target strain.

- In this study, the optimum layer coefficients obtained for the base and sub-base layers show that the materials used in the layers are constructed with well-graded and dry aggregates. In this case, the AASHTO-93 and M-E methods are compatible with each other. However, as the materials used in the base and sub-base layers and moisture contents change, M-E methods must be included in the design solution that takes into account the non-linear behavior of the materials.

- In the empirical pavement design methods related to the road tests, the change of the parameters used in the test affects the design results. It is a remarkable subject of study that the empirical method results are associated with an M-E method since it is not possible to examine a large number of parameters that affect the design of pavement. Thus, M-E design parameters (layer material characterization, contact type, etc.) can be optimized according to the empirical method outputs, and various analyses can be made with M-E design with different loading and environmental conditions.

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