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DEVELOPMENT OF KOREAN PAVEMENT DESIGN GUIDE FOR ASPHALT PAVEMENTS BASED ON THE MECHANISTIC-EMPIRICAL DESIGN PRINCIPLE

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Abstract. Recently, the Korean Pavement Design Guide (KPDG) has been developed based on the mechanistic-empirical design principle as part of the Korea Pavement Research Program (KPRP). This paper presents the detailed information about the input parameters and pavement performance models used in the KPDG for new construction of asphalt pavements. Input parameters for pavement design such as traffic, environment, and materials were characterized by considering the domestic condition. The structural analysis program based on the layer elastic theory has been developed to calculate the critical pavement responses in asphalt pavements. The pavement performance models for fatigue cracking and rutting were developed and calibrated using various types of laboratory testing results and field monitoring data. The procedure for determining the critical evaluation points in calculating the pavement responses has been proposed to reduce the computational time of the design program. The concept of stiffness reduction of asphalt mixtures was also incorporated with the structural analysis program and pavement performance models for more realistic prediction of the pavement distresses.

Keywords: Korean Pavement Design Guide (KPDG), mechanistic-empirical, pavement design, pavement response, pavement performance, stiffness reduction.

1. Introduction

The Korean Pavement Design Guide (KPDG) has been developed based on the mechanistic-empirical design principle with a cumulative damage concept as part of the Korea Pavement Research Program (KPRP). The overall design approach and procedure of the KPDG is similar to the Mechanistic-Empirical Pavement Design Guide (MEPDG) in the United States (Ceylan *et al.* 2009; Witczak *et al.* 2003). However, the input parameters for the pavement design were characterized by considering the domestic situation. Based on the laboratory performance testing results, the pavement performance models for each distress type have been developed in this study. This paper introduces the input parameters and pavement performance models used in the KPDG.

The MEPDG accounts for the effect of both traffic wandering and various axle types on the calculation of the pavement responses and cumulative damage. A total of 30 analysis points in a plan view plane (PVP) is required to estimate the max cumulative damage. In addition, more than ten points along the depth are also needed to evaluate both permanent deformation and fatigue damage. Therefore, a total of 300 evaluation points need to be considered for the analysis. For a 20-year design life, there are, at least, 1200 individual time periods estimated for the default time periods of the MEPDG (i.e. 12 months per year and 5 hour groups per day). The computing time of MEPDG ranges from 30 to 60 min for a single design case when using a typical computer (Khazanovich, Wang 2007).

To reduce computing time, the MEPDG procedure has different approaches for evaluating the fatigue cracking and permanent deformation by considering lateral wandering effects of traffic loading. In case of fatigue cracking, the distribution of damage with traffic wandering can be computed from the damage profile obtained by that has no wandering. In case of permanent deformation, the MEPDG modifies the actual pavement responses for the effects of wandering and uses this modified response for the calculation of the incremental permanent deformations within each layer (Witczak *et al.* 2003). In this study, the procedure for determining the critical evaluation points in calculating the pavement responses has been proposed to reduce the computational time of the program. Through this procedure, a number of evaluation points in a PVP can be reduced from 30 points to less than or equal to 6 points. 2 points are required for the prediction of both rutting and bottom-up cracking, and other points are required for the prediction of top-down cracking. Reduction of evaluation points can help to save the large amount of computational time for the analysis.

Since the stiffness of asphalt mixtures decreases as the damage accumulates with time, the concept of stiffness reduction should be incorporated in the analysis procedure. The dynamic modulus of asphalt mixtures should be updated in every time step based on the accumulated damage level. It has been also found that the MEPDG does not consider the stiffness reduction of asphalt concrete due to a cumulative damage in the version 0.91. The KPDG applies the concept of stiffness reduction of asphalt mixtures to the structural analysis program and pavement performance models for more realistic prediction of pavement performance.

2. Design parameters and performance models

2.1. Environmental effects

It is well known that the environmental condition factors such as temperature and moisture are very important input parameters in the pavement design (Motiejūnas *et al.* 2010; Šiaudinis, Čygas 2007). These factors can significantly affect the material properties and performances in the pavement layers.

A meteorological database of Korea Meteorological Administration (KMA) collected from 68 weather stations for past ten years was utilized to develop the temperature and moisture prediction models in the KPDG. The temperature prediction model is capable of estimating the variations of temperature with time and along the depth in the pavement. The variations of moisture in the subbase and subgrade are estimated using the moisture prediction model developed using field database and regression approach.

2.2. Traffic characterization

The axle load spectrum data for the given vehicle and axle types has been used for characterizing the traffic loading in the KPDG. The axle load spectrum data has been grouped based on the highway classification, annual average daily traffic (*AADT*), and region (urban or rural area). Traffic data collected from several locations in Korea is rearranged into an applicable format that the number of axle load is estimated for every axle load magnitude and axle type within a specific analysis time.

2.3. Material models

2.3.1. Asphalt concrete layer

The dynamic modulus has been selected as an input parameter to characterize the stiffness of asphalt mixtures in the KPDG (Witczak, Fonseca 1996). The indirect tensile testing was conducted to establish a database of dynamic modulus for various types of asphalt mixtures widely used in Korea. The dynamic modulus master curve can be represented by a sigmoid function as shown in Eq (1). A regression analysis has been performed to determine the model coefficients for each mixture type using the laboratory testing database. Detailed information involved in the development of the dynamic modulus model for asphalt concrete can be found in elsewhere (Kwon *et al.* 2007).

$$\log(E^*) = \delta + \frac{\alpha}{1+e^{\beta-\gamma\log(f_r)}},$$

$$\log(f_r) = \log(f) + \log(a(T)), \tag{1}$$

where E^* – dynamic modulus of asphalt mixture, MPa; f_r – frequency of loading at the reference temperature, Hz; f – frequency of loading at a given temperature of interest, Hz; $a(T)$ – shift factor as a function of temperature; α, δ, β, γ – model coefficients.

2.3.2. Subbase and subgrade layer

The stiffness of subbase and subgrade materials was characterized using the stress-dependent resilient modulus (Huang 2003). The repeated load triaxial testing was conducted on unbound materials widely used in Korea to determine the coefficients of stress-dependent soil models. Detailed information involved in the development of the resilient modulus model for unbound materials can be found in elsewhere (Kwon *et al.* 2007). The resilient modulus of subbase can be predicted by the model presented in the following:

$$E = k_1 + k_2\theta, \tag{2}$$

where E – resilient modulus of subbase, MPa; θ – bulk stress ($= \sigma_1 + \sigma_2 + \sigma_3$), kPa; k_1, k_2 – coefficients of model.

Similarly, the resilient modulus of subgrade can be predicted by the following model:

$$E = k_1\theta^{k_2}\sigma_d^{k_3}10^{k_w(w-w_{opt})}, \tag{3}$$

where E – resilient modulus of subgrade, MPa; k_1, k_2, k_3 – coefficients to be determined from regression analysis (Kwon *et al.* 2007); σ_d – the deviator stress, kPa; k_w – (−0.1417) to Coarse – grained soil, k_w – (−0.0574) to fine-grained soil; w – moisture content, %; w_{opt} – optimum moisture content, %.

2.4. Pavement performance model

In mechanistic-empirical based design guide, major pavement distresses are fatigue cracking (top-down and bottom-up), permanent deformation, and thermal cracking (low temperature). Since the thermal cracking is not significant distress observed in Korea, only two distress models (i.e. fatigue and permanent deformation) are considered in the KPDG.

2.4.1. Bottom-Up fatigue cracking model of asphalt mixtures

Bottom-Up (BU) fatigue cracking prediction model of asphalt mixture is expressed as a function of tensile strain and mixture stiffness. The indirect tensile fatigue test is selected to determine the coefficients of the fatigue cracking prediction model (Kwon *et al.* 2004). The BU fatigue cracking model is presented as follows:

$$N_f = 10^M f_1(\varepsilon_0)^{f_2}(S_{mix})^{f_3}, \tag{4}$$

where $M = 4.84(\frac{V_b}{V_b + V_a} - 0.69)$, f_1, f_2, f_3 – model coefficients for asphalt mixture type (Kwon *et al.* 2007); ε_0 – tensile strain of asphalt mixture; S_{mix} – mixture stiffness; V_b – effective binder content, %; V_a – air voids, %.

According to the cumulative damage concept, the stiffness of asphalt mixture decreases as the damage ratio increases. The stiffness reduction factor, S_R, was determined by the indirect tensile fatigue testing for various types of asphalt mixtures. The equation for estimating the S_R is based on the relationships between stiffness of asphalt mixture and number of loading cycles. The stiffness of asphalt mixture values were normalized with the max stiffness value of asphalt mixture and the numbers of loading cycles were normalized with the number of load cycles to failure. Fig. 1 presents the change of the normalized stiffness values with increase of normalized numbers of loading cycles for dense graded mixture with 13 mm nominal max aggregate size and PG64-22 at temperature of 20 °C. Detailed steps involved in this analysis can be found earlier work (Kwon *et al.* 2007). The reduced AC stiffness at any loading cycle can be estimated using the following equations:

$$S_{mix\_R} = S_{mix}\, S_R \tag{5}$$

and

$$S_R = a_R D^3 + b_R D^2 + c_R D + 1, \text{ when } D \le 1,$$
$$S_R = -2.88618 + 2.93253\,(1 - e^{-0.731347D})^{-0.225089}, \text{ when } D > 1, \tag{6}$$

where S_{mix_R} – mixture stiffness of AC after reduction; S_R – stiffness reduction factor; $D = \frac{N}{N_f}$ damage ratio of asphalt layer, %; a_R, b_R, c_R – model coefficients for AC mixture type (Kwon *et al.* 2007).

2.4.2. Top-Down fatigue cracking model of asphalt mixtures

The Top-Down (TD) fatigue cracking model of asphalt mixtures proposed by Lee *et al.* (2003) with a calibration factor and applied in the KDPG is presented as follows:

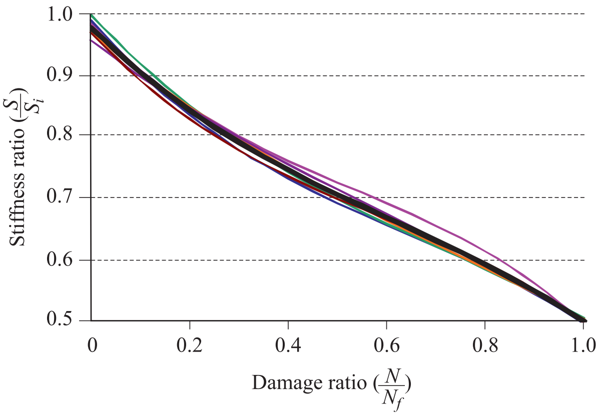

Fig. 1. Stiffness reduction of a dense graded mixture with 13 mm nominal max aggregate size and PG64-22 at 20 °C

$$N_f = \beta \frac{4f}{\sqrt{\pi}} a_1(\alpha_1)^{(0.5+b_1)} \left|E^*\right|^{-2\alpha_1} (\varepsilon_0)^{-2\alpha_1}, \tag{7a}$$

where f – loading frequency, Hz; ε_0 – tensile strain; $\left|E^*\right|$ – dynamic modulus, kg/cm^2; a_1, b_1 – material properties; β – calibration factor of the crack initiation model.

Based on the experimental study, the relationship between α and m can be presented in Eq (7b), while the other model coefficients (a_1 and b_1) are expressed as the functions of strain amplitudes:

$$\alpha = 0.5 + \frac{1}{m}, \tag{7b}$$

$$a = 2.4905(\varepsilon_0)^{-0.1687}, \tag{7c}$$

$$b = 21.301(\varepsilon_0)^{-0.0064}, \tag{7d}$$

where m – the slope of creep compliance versus the time curve in a logarithmic scale.

The calibration factor of the crack initiation model was described as a function of an air void. It was formulated in exponent form as follows:

$$\beta = \beta_1 V_a^{\beta_2}, \tag{8}$$

where V_a – initial air void of asphalt mixture, %; β_1, β_2 – coefficients.

2.4.3. Rutting model of asphalt mixtures

The rutting prediction model of asphalt mixture has been developed by several researchers (Haritonovs *et al.* 2010; Salama *et al.* 2007). The rutting model of asphalt mixture developed in the KPDG is expressed as a function of number of load repetitions, temperature and initial mixture air void. The model coefficients were determined from the triaxial repeated loading tests and then calibrated by Accelerated Pavement Tester's (APT) and Long Term Pavement Performance (LTPP) data. The rutting model can be expressed as follows:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_{AC} a N^b T^c AV^d, \tag{9}$$

where ε_p – accumulated plastic strain at N repetitions of load; ε_r – resilient strain; k_{AC} – correction factor for total AC layer(s) thickness; N – number of load repetitions; T – temperature, °C; AV – initial air void, %; a, b, c, d – coefficients of model for each AC mixture type (Kwon *et al.* 2007).

Rutting models of subbase and subgrade have not been developed yet for the KPDG. Therefore, the models adopted in the MEPDG (Witczak *et al.* 2003) are employed in this study to estimate total surface rut depth.

3. Determination of evaluation points in a plan view plane

The problem of multi-layered elastic system (MLES) subjected to a circular wheel load is considered in polar coordinate system (i.e. (Z, r) coordinate system), while the problem of multiple wheel loads is considered in Cartesian coordinate system (i.e. (x, y, z) coordinate system). Therefore, stresses components in polar coordinate system should be transformed into Cartesian coordinate system before the superposition of multiple axle loads.

The number of evaluation points in polar coordinate system is determined by distances between evaluation points and wheel loads in the Cartesian coordinate system as shown in Fig. 2. The r-coordinate of evaluation points in polar coordinate system is determined by a procedure as follows:

First, all the distances between evaluation points and wheel loads of a certain plane view in the Cartesian coordinate system are sorted ascending. Next, the distance order is based to select the r-coordinate of evaluation points in a polar coordinate system. Several distances having an equivalent length could only be represented by an r-coordinate. By this procedure, the number of selected r-coordinate is always less than or equal the number of distances. The z-coordinate in Cartesian coordinate system needed by the prediction of pavement performance is based to select the z-coordinate of evaluated points in polar coordinate system. Fig. 3 presents a flowchart of determination of evaluation points. The locations of evaluation points are selected to compute the pavement responses for both rutting model and fatigue cracking model.

Fig. 2 shows the locations of 30 evaluation points and six circular wheel loads in a PVP. A total of 10 evaluation points are selected in the x-direction, and three coordinates of 0, 65, and 130 cm were selected in the y-direction.

It is efficient to identify the critical locations in the PVP for each distress type for reducing the evaluation points. In this study, evaluation points are separated into two sets. One set of evaluation points is for the estimation of BU fatigue cracking and rutting. The other set of evaluation points is for the estimation of TD fatigue cracking. Based on following analysis results, the critical points can be selected in a plane for the two sets of evaluation points mentioned above.

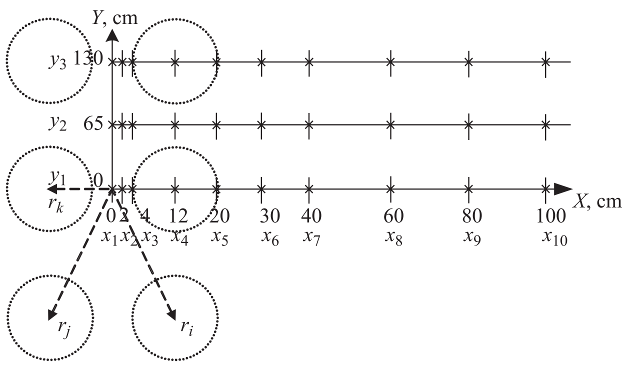

Fig. 2. Evaluation points and wheel load locations in a plan view plane (adapted from NCHRP 1-37A Part 3 2004)

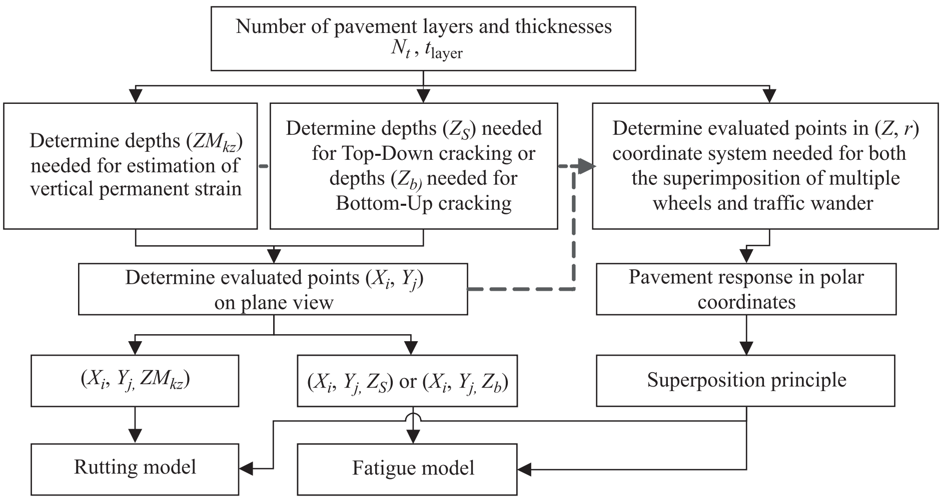

Fig. 3. Preparatory procedure for distress estimation

A pavement section with a total asphalt layer(s) thickness of 30 cm was considered. The asphalt layer was divided into 8 sublayers with the thickness of 4×2.5 cm for the surface layer, and 4×5.0 cm for the base layer. Analysis was performed by the KPDG program for the design period of 10 years. A total of thirty points in the PVP was evaluated for the fatigue damage and rutting.

Figs 4a and 4b show normalized BU fatigue damage accumulated at the bottom of surface layer and base layer, respectively. Figs 4c and 4d show normalized vertical permanent strain accumulated in the first and fourth sublayer of surface layer. Figs 4e and 4f show normalized vertical permanent strain accumulated in the first and fourth sublayer of base layer. It is noted that all the damage values were normalized by max values. It is noticed that $x_1, x_2 \ldots x_{10}$ indicate ten evaluation points in the x axis, while y_1, y_2, and y_3 represent the three evaluation points in the y axis.

As shown in these Figs, the max cumulative damages and vertical permanent strain values can be found in the y_1-coordinate ($y = 0$ cm) indicating that the critical points for the analysis always locate in the y_1-coordinate.

In the x coordinate, the max cumulative BU fatigue damage can be found at the center of a wheel load in the location of x_4 ($x = 12$ cm, $y = 0$ cm) as shown in Figs 4a and 4b. It is also found that the max cumulative vertical permanent strain occur at location of x_4 in the surface layer. In case of base layer, it is observed from Figs 4e and 4f that the cumulative vertical permanent strain values in the middle of the dual wheels, where is the location x_1 ($x = 0$, $y = 0$), are the highest. Therefore, a set of points including both the locations x_4 and x_1 on y_1-coordinate should be selected to determine the critical locations for both BU fatigue and rutting.

For the TD cracking estimation, the x_i points in the y_1-coordinate in Fig. 2 are selected to determine the critical damage while the other y-coordinates (i.e. y_2 and y_3) are neglected. Since the mechanism of TD cracking is affected by many factors such as pavement structures, material characteristics, and environmental conditions, it is difficult to identify the exact critical locations. However, Baladi *et al.* (2003) concluded based on field observations that the first longitudinal TD crack is typically noticed just outside the wheel paths and other longitudinal TD cracks will occur even in the wheel paths with time. Therefore, the points in x axis which are located outside wheel paths (i.e. location x_5, x_6, x_7, x_8, x_9, and x_{10} in Fig. 2) should be selected to determine the critical locations for the estimation of TD cracking potential.

Finally, number of evaluated points in a PVP can be reduced from thirty points to less than or equal to six points. Two points are required for the prediction of both rutting and BU cracking. And other six points are required for the prediction of top-down cracking. By this approach, the large amount of computational time for the analysis can be reduced.

4. Analysis procedure considering stiffness reduction

Since the stiffness of asphalt mixtures decreases as the damage accumulates with time, the concept of stiffness reduction should be incorporated in the analysis proce-

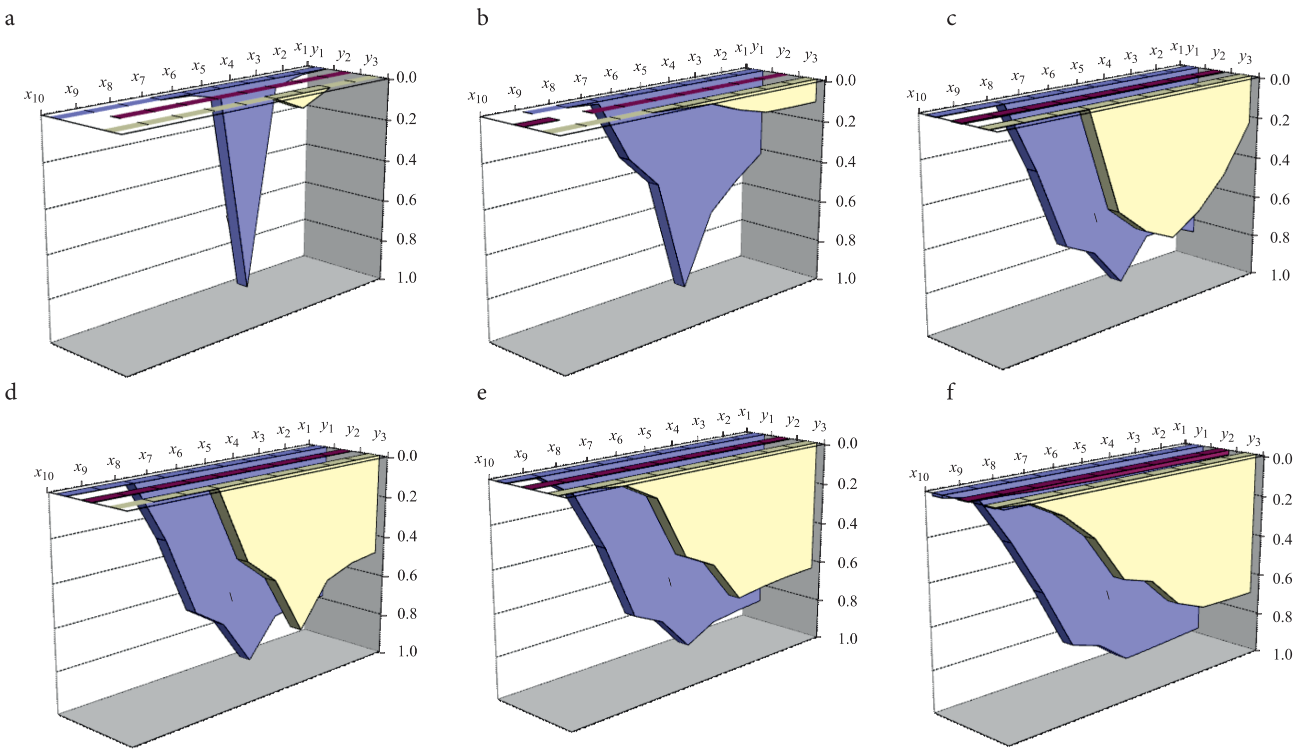

Fig. 4. Normalization of damage and permanent strain predicted at various depths: a, b – cumulated bottom-up fatigue damage; c, d, e, f – cumulated vertical permanent strain

dure. The dynamic modulus of asphalt mixtures should be updated in every time step based on the accumulated damage level.

A case study has been done to consider the influence of stiffness reduction on the estimation of pavement performance. Fig. 5 presents the comparison of the max BU fatigue cracking damage obtained by KPDG and MEPDG for 20-year design life. The thicknesses of asphalt layer for this comparison are 10, 20, 25, and 30 cm.

It is observed from Fig. 5a that the accumulated damage of KPDG increases rapidly after 80th month, while the accumulated damage of MEPDG increases at constant rate from 1st month to 240th month for 10 cm of thickness. The rapid increase of accumulated damage is due to the stiffness reduction of asphalt mixtures.

Fig. 5b shows no difference in percent of damage between KPDG and MEPDG in case of AC thickness of 25 and 30 cm. However, the difference in percent damage between KPDG and MEPDG can be observed in case of asphalt layer thickness of 20 cm. Based on this observation, it can be concluded that the thinner the asphalt layer thickness the larger the observed difference in percent damage between KPDG and MEPDG.

Another approach is proposed in this case, which is developed based on the regression equation. Besides the predicted distress results of the first year, pavement predicted distresses of at least three other sequential years are required to build up regression equations. The established

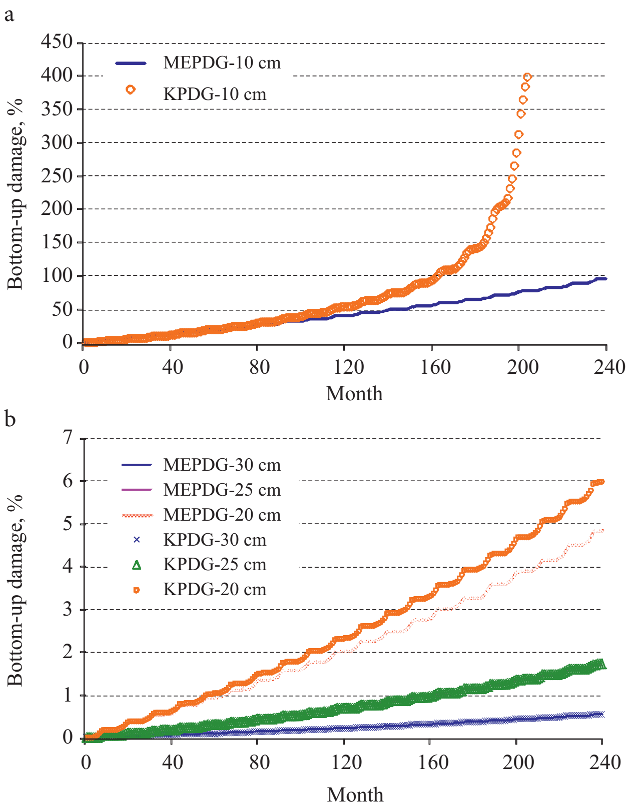

Fig. 5. Max bottom-up damage prediction by both KPDG and MEPDG in cases of various AC thicknesses: a – AC thickness of 10 cm; b – AC thickness of 20, 25, and 30 cm

regression equations will be used to estimate distress for other remaining years.

In addition, it needs to control the error of estimation of pavement distress because the error may be accumulated through entire pavement design life by the approximation of regression equation. Data of pavement predicted distress for more than one year is required to create control points. The number of control points is decided based upon the pavement design life and the expected tolerance of error. In the best case, only computing time for the pavement analyses within 5-years in pavement design life are enough to predict pavement distresses for the pavement design life of 20 years. They include four years of the establishment of regression equations, and one year of a control point. An approximation includes two separated sets of regression equation:

1. Regression equations are needed to predict distress at the beginning of every year in pavement design life. These equations are presented in Eq (10) and Eq (11) for the percent of fatigue damage and rut depth, respectively;
2. Regression equations are needed for the monthly interpolation of distress within a considering year (Eq (12)).

A 2nd-order polynomial is used to approximate accumulated damage versus time in case of fatigue damage prediction, as follows:

$$y = a_1 t^2 + a_2 t + a_3, \tag{10}$$

where y – approximate value of percent of fatigue damage, %; t – time, year; a_1, a_2, a_3 – coefficients of regression equation.

In case of rutting, the approximation equation has the following form:

$$y = a_1 t^{a_2}, \tag{11}$$

where y – approximate value of rut depth, cm; t – time, year; a_1, a_2 – coefficients of regression equation.

The accumulated distress within i^{th} month of considering year (t) can be interpolated by the following equation:

$$b_i = y(t) a_i, \tag{12}$$

where $y(t)$ – distress increase estimated by Eq (10) or Eq (11); i – month index (i = 1÷12); t – time, years; a_i – a distribution factor of i^{th} month (the second year is selected to determine these distribution factors).

Fig. 6 shows the comparison results between the analyses performed through a long period of pavement design life and the approximations performed by regression equations for various AC layer thicknesses.

Figs 6a and 6c show a gap between the approximations and full analyses in case of AC thickness of 10 cm (thin AC layer). However, the approximate procedure can control the accumulated errors with the more control data points.

Figs 6b and 6d show other cases of approximation (AC thickness of 20, and 30 cm). The figures present good agreement of approximation. The accumulated errors are less than 5%. It indicates that the proposed approach is a reasonable approach.

By this approach, computing time spent for entire pavement performance prediction can be reduced about 75% compared to a full analysis.

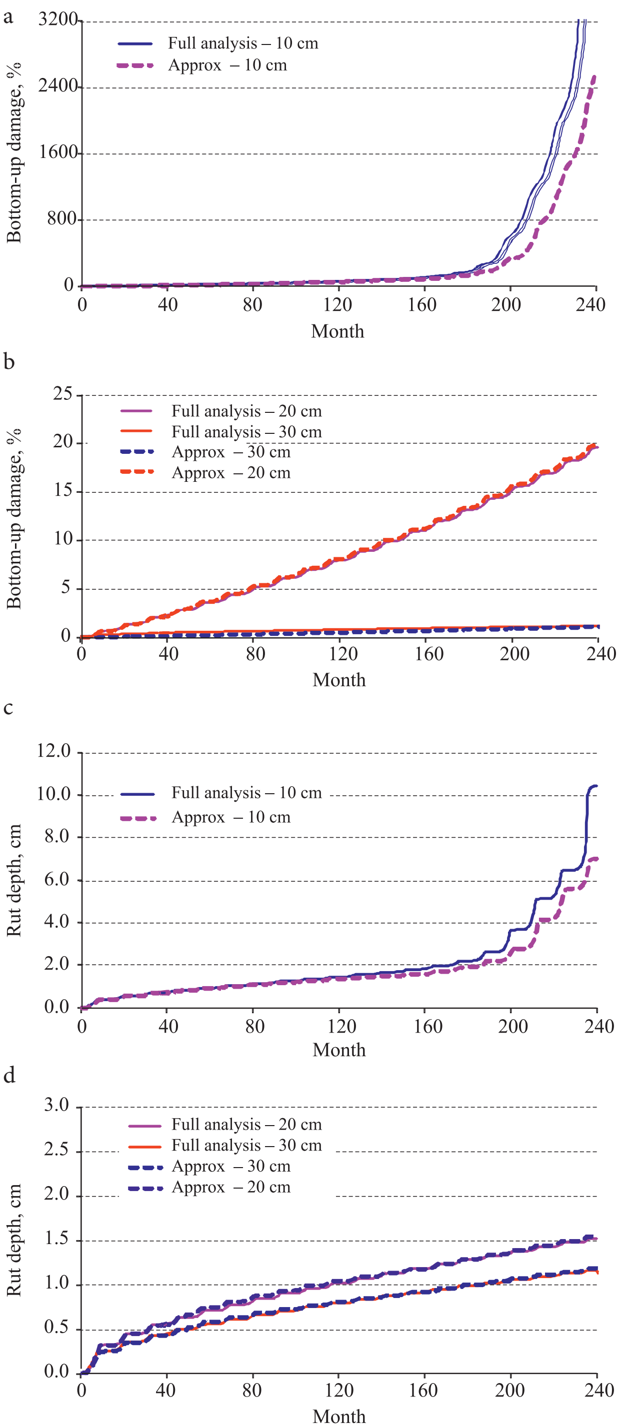

Fig. 6. Comparisons of full analysis and approximation: a, b – bottom-up damage; c, d – AC rutting

5. Conclusions

As part of Korean Pavement Research Program (KPRP), the Korea Pavement Design Guide (KPDG) has been developed based on the mechanistic-empirical design principle in this study. Details on the input parameters and pavement performance models for the KPDG have been presented.

The analysis procedure to reduce the computational time has been proposed by decreasing the evaluation points in the PVP. It is found from this study that this method enables to reduce the evaluation points from thirty to less than or equal to six points in PVP for estimating the fatigue cracking and rutting potentials. The techniques for grouping the time period based on the regularity of climate and characteristics of axle load magnitudes can also help to reduce the consuming time of analysis.

The proposed procedure is capable of considering the stiffness reduction of asphalt mixture in its analysis process. An efficient approximate method can be employed in the procedure to reduce computing time in case the stiffness reduction factor model is considered.

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References

Baladi, G. Y.; Schorsch, M.; Svasdisant, T. 2003. *Determining the Causes of Top-Down Cracks in Bituminous Pavements*. Report No. RC-1440. 255 p.

Ceylan, H.; Coree, B.; Gopalakrishnan, K. 2009. Evaluation of the Mechanistic-Empirical Pavement Design Guide for Implementation in Iowa, *The Baltic Journal of Road and Bridge Engineering* 4(1): 5–12. doi:10.3846/1822-427X.2009.4.5-12

Haritonovs, V.; Smirnovs, J.; Naudžuns, J. 2010. Prediction of Rutting Formation in Asphalt Concrete Pavement, *The Baltic Journal of Road and Bridge Engineering* 5(1): 38–42. doi:10.3846/bjrbe.2010.05

Huang, Y. H. 2003. *Pavement Analysis and Design*. 2nd edition. Prentice Hall. 792 p.

Khazanovich, L.; Wang, Q. 2007. MnLayer: High-Performance Layered Elastic Analysis Program, *Transportation Research Record: Journal of the Transportation Research Board* 2037: 63–75. doi:10.3141/2037-06

Kwon, S. A.; Cho, Y. H.; Hwang, S. D. 2004. *Development of Korean Pavement Design Guide Phase 1*. Report of Korean Pavement Research Project. Draft Report KPRP-L-04. Publication 11-1500000-000912-14. Goyang City, South Korea.

Kwon, S. A.; Cho, Y. H.; Hwang, S. D. 2007. *Development of Korean Pavement Design Guide Phase 4*. Report of Korean Pavement Design Guide Project. Draft Final Report KPRP-G-07. Goyang City, South Korea.

Lee, H. J.; Kim, Y. R.; Lee, S. W. 2003. Fatigue Life Prediction of Asphalt Mixes Using Viscoelastic Material Properties, *Transportation Research Record* 1832: 139–147. doi:10.3141/1832-17

Motiejūnas, A.; Paliukaitė, M.; Vaitkus, A.; Čygas, D.; Laurinavičius, A. 2010. Research on the Dependence of Asphalt Pavement Stiffness upon the Temperature of Pavement Layers, *The Baltic Journal of Road and Bridge Engineering* 5(1): 50–54. doi:10.3846/bjrbe.2010.07

Salama, H. K.; Haider, S. W.; Chatti, K. 2007. Evaluation of New Mechanistic–Empirical Pavement Design Guide Rutting Models for Multiple-Axle Loads, *Transportation Research Record: Journal of the Transportation Research Board* 2005: 112–123. doi:10.3141/2005-13

Šiaudinis, G.; Čygas, D. 2007. Determination of Seasonal Effects on the Structural Strength of Asphalt Pavements, *The Baltic Journal of Road and Bridge Engineering* 2(2): 67–72.

Witczak, M. W.; Fonseca, O. A. 1996. Revised Predictive Model for Dynamic (Complex) Modulus of Asphalt Mixtures, *Transportation Research Record: Journal of the Transportation Research Board* 1540: 15–23. doi:10.3141/1540-03

Witczak, M.; Zapata, C. E.; Von Quintus, H. 2003. Input Data for the Calibration and Validation of the Design Guide for New Constructed Flexible Pavement Sections, in *Guide for Mechanistic–Empirical Design of New and Rehabilitated Pavement Structures*, ARA, Inc., ERES Division, Champaign, IL: 1–50.

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