SENSITIVITY ANALYSIS IN FLEXIBLE PAVEMENT PERFORMANCE USING MECHANISTIC EMPIRICAL METHOD (CASE STUDY: CIREBON–LOSARI ROAD SEGMENT, WEST JAVA)

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ABSTRACT

Cirebon – Losari flexible pavement which is located on the North Coast of Java, Indonesia, is in the severe damage condition caused by overloading vehicles passing the road. The need for developing improved pavement design and analysis methods is very necessary. The increment of loads and quality of material properties can be evaluated through Mechanistic-Empirical (M-E) method. M-E software like KENLAYER has been developed to facilitate the transition from empirical to mechanistic design methods. From the KENLAYER analysis, it can be concluded that the effect of overloading to the pavement structure performance is difficult to minimize even though the first two layers have relatively high modulus of elasticity. The occurrence of 150%, 200%, and 250% overloading have a very significant effect in reducing 84%, 95%, and 98% of the pavement design life, respectively. For the purpose of increasing the pavement service life, it is more effective to manage the allowable load.

Keywords: flexible pavement, overloading, mechanistic empirical method, KENLAYER.

INTRODUCTION

A. Background

Pantura national highway, which is located on the North Coast of Java, Indonesia, is in the severe damage condition caused by overloading vehicles passing the road. Pantura deteriorates faster than it should be because the average overload has reached 100% over the standard regulation (Department of Public Works, 2008).

Almost all of the national highways in Indonesia, including Pantura, are flexible pavement. Over a few decades, the design of this flexible pavement has been based on empirical method, American Association of State Highway and Transportation Officials (AASHTO) guides for pavement design (AASHTO, 1993). The 1993 AASHTO guide is based solely on the results of the AASHO Road Test from the late 1950s.

The condition of traffic volume and materials in the AASHO Road Test were relatively different from the condition of highways nowadays. Moreover, since there has been a phenomenon of overloading in many countries and improvement of material properties quality in flexible pavement design which are not considered in AASHTO 1993, the need for developing improved pavement design and analysis methods is very necessary.

The increment of loads and quality of material properties can be evaluated through Mechanistic-Empirical (M-E) method which is based on elementary physics and determines pavement response to wheel loads or environmental condition in terms of stress, strain, and displacement. M-E software like KENLAYER has been developed to facilitate the transition from empirical to mechanistic design methods.

B. Problem Statement

As a part of of Pantura, Cirebon – Losari road segment which has a relatively high volume of traffic, around 88.9 million ESALs (equivalent single axle loads) on 2018, has pavement deterioration problem. This problem is occurred mostly because of the overloading of the heavy truck. Based on the result of the overloading survey under the heavy truck loads conducted by Directorate General of Highway, Department of Public Works (2008), on Cirebon – Losari road segment which is part of Pantura, the overloading rate has reached 200%.

The 1993 AASHTO guides that are normally adopted to design the flexible pavement in Indonesia were developed based on AASHO Road Test that was trafficked with less than 2 million ESALs. If it is compared with the actual traffic on Cirebon – Losari road segment, this traffic volume is considerably low. According to Lister et al. (1982) if the traffic volume exceeds 10 million ESALs, the implementation of the empirical method in the design is inaccurate.

To deal with these issues, it is better to implement the mechanistic empirical method in the flexible pavement design. The advantages of the M-E design over the empirical methods are it tolerates (Timm, et al., 1998):

1. Better utilization and characterization of available materials,
2. Improved performance predictions,
3. Relation of material properties to actual pavement performance,
4. Better definition of the existing pavement layer properties.

C. Objective

The objectives of this study are:

1. To implement the M-E method in flexible pavement design with consideration of traffic loading and material properties.
2. To investigate the flexible pavement performance due to overloading and variation in material properties.
3. To predict the design life of the flexible pavement due to overloading and variation in material properties.
4. To perform the sensitivity analysis of overloading and variation in material properties.

D. Scope and Limitation

The scope and limitation of the study are:

1. This study is focused on flexible pavement of Cirebon – Losari road segment (Km 18+200 – 21+723), West Java, Indonesia.
2. Material properties and traffic data are based on the secondary data obtained from the Directorate General of Highway, Ministry of Public Works for Cirebon – Losari road segment project.
3. Data of overloading is based on the data surveyed by Directorate General of Highway, Ministry of Public Works.
4. The overloading is considered as the increment of Vehicle Damage Factor which is based on 80 kN single axle load (ESAL).
5. The sensitivity analysis of changing in material properties is limited only on the two most top layers

LITERATURE REVIEW

A. Flexible Pavements

Flexible pavements are pavements constructed with bituminous and granular materials. These types of pavements are so named since the total pavement structure deflects/bends under traffic loading. Flexible pavements are layered systems that can be analyzed with Burmister's layer theory (Burmister, 1943).

Flexible pavements structure may be composed of several layers of material with great thickness for optimally transmitting load to the subgrade. These layered systems have high quality materials on the top where stresses are high and low quality materials at the bottom.

B. Empirical Methods

The empirical AASHTO method (AASHTO, 1993), which is based on the AASHO Road Test conducted from 1958-1960 in Ottawa, Illinois, is the most widely used pavement design method today.

This design originated the concept of pavement failure based on the deterioration of ride quality or serviceability over time or application of traffic loading as perceived by the user. The traffic loading was introduced in terms of a single statistic known as the 18-kip equivalent single axle load (ESAL).

Empirical equation is used to correlate pavement characteristics with pavement performance. The Equation 1 is the 1993 AASHTO Guide basic design equation for flexible pavements that is broadly used:

$$log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times log_{10}(SN+1) - 0.20 + \frac{log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times log_{10}(M_R) - 8.07 \quad (1)$$

where: W_{18} : predicted number of 80 kN (18,000 lb.) ESALs

Z_R: normal standard deviation

S_o : combined standard error of the traffic prediction and performance prediction

SN : Structural Number (an index that is indicative of the total pavement thickness required)

$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 + ...$

a_i : i^{th} layer coefficient

D_i : i^{th} layer thickness (inches)

m_i : i^{th} layer drainage coefficient

ΔPSI : difference between the initial design serviceability index, p_o, and the design terminal serviceability index, p_t

M_R : subgrade resilient modulus (in psi)

C. Mechanistic Empirical Methods

Mechanistic Empirical (M-E) design method is a logical engineering approach that has been widely used to replace the empirical AASHTO design procedure (AASHTO 1993). The main advantage of an ME design method is that the analysis is based on pavement fatigue and deformation characteristics of all layers, rather than only on the pavement's surface performance (ride quality). It is based on the mechanistic of materials that relates traffic load to pavement response, such as stress and strain.

Mechanistic empirical computer program can be used to run the calculation of stress, strain, and deflection in mechanistic empirical methods. By using this computer program, all the pavement reactions due to the load repetition can be determined more accurately, close to the actual condition.

KENLAYER computer program applied only to flexible pavement has been used for determining the damage ratio using distress models. It is the solution for an elastic multilayer system under a circular loaded area by superimposing for multiple wheels, applying iteratively for non-linear layers, and collocating at various times for viscoelastic layers.

The distress models evaluated in KENLAYER are fatigue cracking and permanent deformation or rutting. In designing the flexible pavement, the most critical consideration is strain due to cracking and rutting. The fatigue cracking is caused by the horizontal tensile strain (ε_t) at the bottom of the asphalt layer and the permanent deformation or rutting is caused by vertical compressive strain (ε_c) on the surface of subgrade.

The fatigue cracking models are developed from Miner's cumulative damage concept. The main difference in the various design methods is the transfer functions associate the HMA tensile strains (ε_t) to the allowable number of load repetitions (N_f). The allowable number of load repetition can be computed using Equation 2.

$$N_f = f_1(\varepsilon_t)^{-f_2}(E)^{-f_3} \quad (2)$$

where: ε_t : horizontal tensile strain at the bottom of the HMA layer

E_{AC} : modulus of elasticity of the HMA

f_1, f_2, f_3 : constants obtained by calibration

The permanent deformation models are used to control the vertical compressive strain on the top of the subgrade. The allowable number of load repetition (N_d) to limit rutting is related to the vertical compressive strain (ε_c) on top of the subgrade by Equation 3.

$$N_d = f_4(\varepsilon_c)^{-f_5} \tag{3}$$

where: ε_c : vertical compressive strain at the top of subgrade layer
N_d : number of load repetition to failure
f_4, f_5 : calibrated values using predicted performance and field observation

The damage ratio is the ratio between the predicted and allowable number of repetition. It is computed for each load group in each period and summed over the year by Equation 4.

$$D_r = \sum_{i=1}^{p}\sum_{j=1}^{m}\frac{n_{i,j}}{N_{i,j}} \tag{4}$$

where: D_r : damage ratio at the end of a year
$n_{i,j}$: predicted number of load repetitions for load *j* in period *i*
$N_{i,j}$: allowable number of load repetitions for load *j* in period *i*
p : the number of periods in each year
m : number of load groups

The design life is computed through Equation 5 and calculated for fatigue cracking and for permanent deformation, and the one with a shorter life controls the design.

$$Design\ life = \frac{i}{D_r} \tag{5}$$

METHODOLOGY OF THE STUDY

A. Data Collection

This study is carried out to figure out the effect of overloading and utilization of various qualities of material properties on the performance and design life of the pavement. The secondary data used in this study are collected from the Ministry of Public Works. There are two types of data applied for the analysis, those are:

1. Data of Cirebon – Losari flexible pavement design
2. Data of Cirebon – Losari overloading survey

B. Method of Data Analysis

The method in analyzing the data used in this study is mechanistic empirical method. The secondary data are evaluated in several scenarios for the sensitivity analysis due to variation in overloading and material properties for determining the pavement performance and design life. A computer program KENLAYER is used to analyze the distress on the flexible pavement layer.

The input for analysis consists of two main parameters: traffic loading and material properties, which are keyed in KENLAYER using menu: LAYERINP. The SI unit system is used since it is normally applied in Indonesia.

The structural analysis of flexible pavement for KENLAYER is based on the Burmister layer theory. The damage analysis is performed for the fatigue cracking and permanent deformation. Then, the analysis is used further to calculate the design life and made only in one period, since the environment is assumed to be constant.

C. Traffic Loading

Load configuration has an effect on the stress distribution and deflection within a pavement. Many trucks have dual wheels which guarantee that the contact pressure is within the limits. In this study for simplification of the analysis, the dual wheels are converted into an equivalent 80-kN single axle load (ESAL). Others items of information about the load required in LAYERINP are:

1. The contact radius of circular loaded area (CR),
2. Contact pressure on circular loaded area (CP),
3. Center to center spacing between two dual wheels along the y axis (YW), and
4. Number of points in x and y coordinates to be analyzed under multiple wheels (NPT).

The contact area is important to be determined so the axle load can be assumed to be uniformly distributed. In this study, only wheels on one side (the outer wheel path) need to be considered and each tire is assumed to have circular contact area. The tire spacing is assumed with a typical distance between dual tires of 35 cm (Timm et al., 1998).

The tire radius of the contact area for commercial vehicles is 10.74 cm based on PCA (1984). The tire pressure on contact area is suggested by Taesiri and Jitareekul (2003) in the range of 550-700 kPa. Since the load is analyzed using 80-kN single axle load (ESAL), then the tire pressure is calculated using Equation 6.

$$P = \frac{F}{A} = \frac{40}{2\pi \times 0.1074^2} = 552 \cong 550\ kPa \quad (6)$$

In this study, the load information that keyed in LAYERINP is shown in Table 1 and the stress points in x and y coordinates are shown in Figure 1.

Table 1. Load information

CR	CP	YW	NPT
10.74 cm	550 kPa	35 cm	3

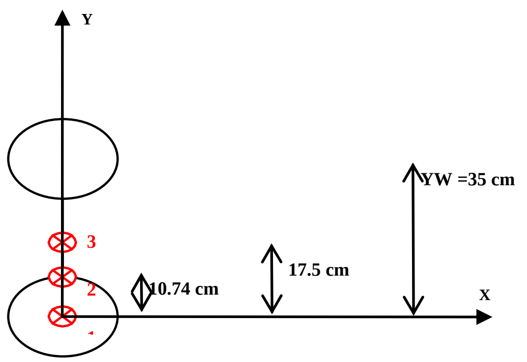

Figure 1. The tire spacing and location of stress points (Huang, 1993)

The initial traffic loads used for the analysis in this study are based on the traffic data Cirebon – Losari flexible pavement design. The traffic data are used to compute the number of ESALs based on the 1993 AASHTO guide. The data required for calculating the ESALs are:

a. Average daily traffic (ADT),
b. The equivalent axle load factor/vehicle damage factor (EALF/VDF),
c. The directional split (D) (L), and
d. The growth factor (G) (Y).

The VDF used in this study is calculated based on Equations 7 to 10 introduced by Department of Public Works (2008).

$$VDF_{single\ axle\ single\ tires} = \left(\frac{P}{5.4}\right)^4 \quad (7)$$

$$VDF_{single\ axle\ dual\ tires} = \left(\frac{P}{8.16}\right)^4 \quad (8)$$

$$VDF_{tandem\ axles} = 0.086\left(\frac{P}{8.16}\right)^4 \quad (9)$$

$$VDF_{tridem\ axles} = 0.053\left(\frac{P}{8.16}\right)^4 \quad (10)$$

In this study, the growth factor is calculated using the Equation 11 developed by the Asphalt Institute (AI, 1981) and the AASHTO design guide (AASHTO, 1986). They suggested the use of traffic over the entire design period to calculate the total growth factor. The growth rate (r) used in this study is 6% and the design period (Y) is 10 years.

$$(G)(Y) = \frac{(1+0.06)^{10} - 1}{0.06} \quad (11)$$

The calculations of the number of ESALs follow the Equation 12 (Huang, 1993) where T and Tf are stand for the percentage of truck and truck factor, respectively. The result of these computations is total number of load repetitions and it is applied as the input in LAYERINP for the damage analysis.

$$ESAL = (ADT)_0(T)(T_f)(G)(D)(L)(365)(Y) \quad (12)$$

D. Material Properties

The main properties used in this study are the modulus elasticity, the Poisson's ratio, and the unit weight of each layer. The material properties for each layer can be seen in the Figure 2.

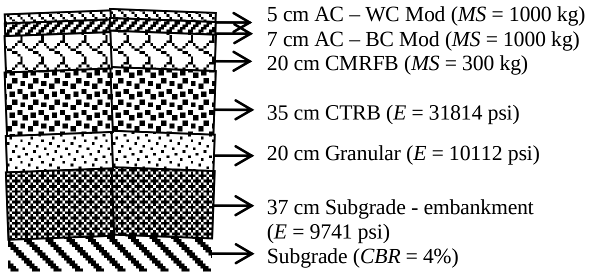

Figure 2. Typical cross section of Cirebon – Losari road segment

As can be seen in Figure 2, the characteristic of the material for AC – WC Mod and AC – BC Mod are similar, therefore for the simplification in this study, they are combined as one layer Asphaltic Concrete (AC) with 12 cm thickness. Consequently, there are six layers analyzed differently as linear viscoelastic, linear elastic, or nonlinear elastic layer.

The thickness of each layer is stored in LAYERINP. The modulus of elasticity, Poisson's ratio and unit weight of each layer keyed in LAYERINP can be seen in Table 2.

Table 2. Material properties

No	Materials	Modulus of Elasticity (MPa)	Poisson's Ratio	Unit Weight (kN/m^3)
1	Asphaltic Concrete (AC)	2,758	0.35	22.8
2	Cold Milling Recycling Foam Bitumen (CMRFB)	1,241	0.35	21.2
3	Cement Treated Recycling Base (CTRB)	220	0.35	21.2
4	Granular	70	0.35	21.2
5	Subgrade (embankment)	67	0.35	21.2
6	Subgrade	33	0.45	19.6

Source: Department of Public Works (2008), Timm, et al. (1998)

The viscoelastic is applied to analyze the AC layer. The behavior of asphalt depends on the time of loading, so the theory of viscoelasticity is normal to be used. The load duration is 0.1 sec for moving load 40 mph. The method for characterizing viscoelastic material is by specifying the creep compliances.

The reference temperature for the creep compliance used as the input in LAYERINP is assumed to be the same as the initial design that is 25°C. The generalized model for creep compliance at reference temperature 25°C is developed from Huang (1993) and expressed in Equation 13.

$$D(t) = \frac{1}{E}(1 - e^{-0.833t}) \qquad (13)$$

Creep compliances are measured at 11 varieties of times of 0.001, 0.03, 0.01, 0.03, 0.1, 0.3, 1, 3, 10, 30 and 100 seconds (FHWA, 2002). KENLAYER specifies retardation times of 0.01, 0.03, 0.1, 1, 10, 30, and ∞ seconds since moving load usually has a short duration.

The layers that are assumed to be analyzed as linear elastic are the CMRFB and CTRB layers. The recycling foam bitumen and cement treated material are normally considered as linear elastic with a constant modulus of elasticity. The required input in LAYERINP is the modulus of elasticity of each layer.

The granular, subgrade (embankment), and subgrade layers are analyzed as nonlinear elastic layer. The elastic modulus of these layers varies with level of stress. In this study, the type material of the granular and subgrade (embankment) are assumed to be sand-aggregate blend and the subgrade is assumed to be very soft soil.

There are several constants required in analyzing the nonlinear elastic: K0, K1, K2, K3, K4, E_{min} and E_{max}. The coefficient of earth pressure at rest (K0) recommended by Monismith and Witczak is 0.8 (Huang, 1993). Finn et al. (1986) suggested that the range of constant nonlinear coefficient of granular layer (K1) is from 3,200 to 8,000 psi (22,080 to 55,200 kPa) and the non linear exponent (K2) is 0.6. Based on the recommendation from Rada and Witczak (1981), K2 for sand-aggregate blend is 4,350 psi (30,450 kPa). The values of nonlinear constants used in this study are shown in Table 3.

Table 3. Nonlinear constants of nonlinear elastic layer

No	Nonlinear Layer	K_0	K_1	K_2	K_3	K_4	E_{max}	E_{min}
		(kPa)						
1	Granular	0.8	30.0	0.6	-	-	-	-
2	Subgrade (embank ment)	0.8	27.0	0.6	-	-	-	-
3	Subgrade	0.8	6.9	42. 8	7.7	0	39.1	6.9

In addition, other inputs required for the analysis are the angle of internal friction of granular materials (PHI) and z coordinate of points (ZCNOL). Since the granular base in not subdivided into a number of layers and the pavement has subgrade with a modulus higher than 6.9 MPa, then a PHI of 50 is suggested (Huang, 1993).

Stress points must be located to determine the modulus of elasticity of each nonlinear layer. Since only the maximum stresses, strains, or deflection are required, the stress point should be located under the center of two dual wheels, with XPTNOL = 0, YPTNOL = YW/2 = 17.5 cm, and SLD = 0.

Even though the modulus of elasticity can be calculated at any point in a nonlinear granular layer, it is recommended that the z coordinate is located at the middepth of each layer and for the nonlinear subgrade, a point of 1 inch below the subgrade is used to determine the vertical compressive strain on the top of subgrade. The location of ZCNOL used in this study can be seen in Table 4.

Table 4. Location of ZCNOL

No	Nonlinear Layer	ZCNOL (cm)
1	Granular	77
2	Subgrade (embankment)	105.5
3	Ground	126.5

E. Damage Analysis

The damage analysis is completed for both fatigue cracking and permanent deformation. For the fatigue cracking it is based on the horizontal tensile strain at the bottom of AC layer or layer 1 and for the permanent deformation based on the vertical compressive strain on the top of the subgrade or layer 6.

These critical distresses are used to determine the number of allowable loads before fatigue cracking or rutting. The tolerable number of load repetitions for fatigue is computed by using Equation 14 while Equation 15 is used for rutting. The damage coefficients used as input in LAYERINP were developed by the Asphalt Institute.

$$N_f = 0.414\ (\varepsilon_t)^{3.291}(E)^{0.854} \tag{14}$$

$$N_d = 1.365\mathrm{E}-9\ (\varepsilon_c)^{4.477} \tag{15}$$

ANALYSIS

A. Data Analysis

Data required for this research were the parameters used in the design of Cirebon – Losari Road Segment, i.e. the traffic loading and material properties. For the sensitivity analysis, it includes the traffic overloading that used to happen on that segment and the material properties normally applied in the construction of flexible pavement in Indonesia.

Data taken from the design parameter of flexible pavement of Cirebon – Losari Road Segment were used as the base case of the sensitivity analysis. The data of material properties shown in Table 5 and the cumulative equivalent single axle load for the 10 years design life (2008 – 2018) = 8.89 x 107 were used as the input in LAYERINP. The traffic data shown in Table 6 with growth rate equal to 6% was processed to compute the variation for the sensitivity analysis of traffic loading.

Table 5. Data of material properties

No	Layer Type	MS (kg)	CBR (%)	E (MPa)	Thickness (cm)
1	Asphaltic Concrete (AC)	1,000	-	2,758	12
2	Cold Milling Recycling Foam Bitumen (CRFMB)	300	-	1,241	20
3	Cement Treated Recycling Base (CTRB)	-	115	220	35
4	Granular	-	10.5	70	20
5	Subgrade (embankment)	-	10.5	67	37
6	Ground	-	4	33	∞

Table 6. Data of traffic loading

No	Vehicle Type	Load (ton)	Directional Split	ADT (2007)
1	Car	2	0.3	9,194
2	Utility	8	0.3	7,843
3	Bus	10.5	0.45	6,534
4	Light Truck (1-2)	8	0.45	2,709
5	Medium Truck (1-2)	13	0.45	1,730
6	Heavy Truck (1-1-2.2)	27	0.45	4,221
7	Truck (1-2.2-2.2)	44	0.45	1,997
8	Truck (1-2.2-2.2.2)	56	0.45	418
Total				34,646

B. Sensitivity Analysis in Traffic Loading Variation

According to the data obtained, the loading variations that are calculated in this research only consist of four types of truck; they are medium truck (13 tones), heavy truck (27 tones), truck (44 tones), and truck (56 tones). The loading variation was determined by the incremental of the actual load from the standard load regulated by the Ministry of Transportation.

The calculation of the incremental of standard load and the percentage of distribution for each axle (front axle, middle axle and rear axle) can be seen in Table 7. For the heavy truck (27 tones), truck (44 tones), and truck (56 tones), the calculation of the percentage were simplified by averaging the values.

From the calculation shown in Table 7 below, the range of truck overloading is from 182% to 224%. As the result, based on the range mentioned before, in this research, there will be three types of variation in truck loading, those are 150%, 200%, and 250%.

Since the traffic loading data were used as the input in LAYERINP, for simplification the data were converted into Equivalent Single Axle Load (ESAL) using the AASHTO method. The overloading is modeled as an increment of Vehicle Damage Factor (VDF). Because of that, the overloading was calculated as an increment in the volume of traffic.

Table 7. Incremental of standard load

Truck Type		2 axle	4 axle	5 axle	6 axle	Average (2, 3, 4)
Standard (ton)		13	27	32	40	
Actual (ton)		29.13	57.38	65.4	72.83	
Increment (ton)		16.13	30.38	33.4	32.83	
% Increment		124	113	104	82	
Front Axle	Standard	5	5	5	5	
	Actual	8.55	7.04	6.68	7.25	
	Increment	3.55	2.04	1.68	2.25	
	% Increment	22	7	5	7	6
Middle Axle	Standard		7	7	15	
	Actual		15.59	13.16	21.45	
	Increment		8.59	6.16	6.45	
	% Increment		28	18	20	22
Rear Axle	Standard	8	15	20	20	
	Actual	20.58	34.75	45.55	44.12	
	Increment	12.58	19.75	25.55	24.12	
	% Increment	78	65	77	73	72

From the mechanistic empirical analysis using KENLAYER, with all the input data mentioned before, the results are summarized in the following Table 8 and Figure 3. Based on Table 8 and Figure 3, the variation in truck loading influences the design life, the higher the overloading, the lower the design life. If the traffic loading was increased to 150%, 200%, and 250%, the design life is reduced to 16%, 5%, and 2%, respectively.

Table 8. Result of sensitivity analysis in truck loading variation

No	Variation of Truck Loading (%)	Cumulative ESAL (10^6)	ε_t	ε_c	N_f	N_d	Design Life (year)
1	100	88.9	-1.259E-05	4.948E-05	8.877E+08	2.578E+10	10
2	150	551.79					1.61
3	200	1,915.56					0.46
4	250	5,031.81					0.18

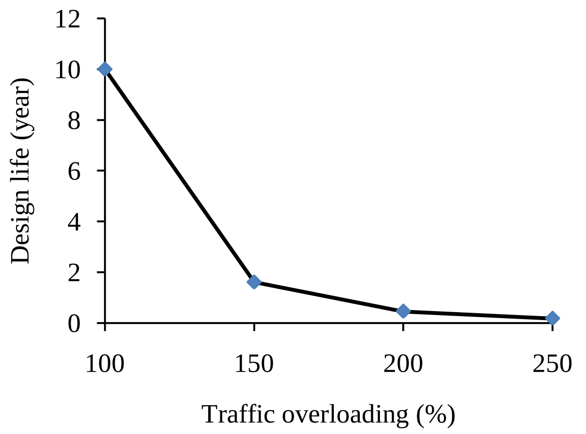

Figure 3. Effect of traffic overloading to the design life

The horizontal tensile strain at the bottom of asphalt layer and vertical compressive strain at the top of subgrade layer were not impacted since the traffic loading was considered as the volume of traffic or predicted number of load repetitions for analyzing the damage analysis. Thus, it also did not affect the number of repetitions of a given load to failure for both rutting and cracking. So, for this type of analysis, the design life was the only result that needs to be considered.

C. Sensitivity Analysis in Material Properties

The variations in material properties were based on the data obtained from the literature review and limited only in the two most top layers that consist of Asphalt Concrete and Cold Milling Recycling Foam Bitumen (CMRFB). The material properties of each layer were varied based on the upper bound and lower bound of the modulus of elasticity (E) and as the result there were four variations for the sensitivity analysis in material properties. The scenario of the variation can be seen in Table 9.

Table. Scenario for material properties variation

No	Layer	Modulus of Elasticity(MPa)	
		Upper Bound	Lower Bound
1	AC	5,000	2,000
2	CMRFB	2,500	1,000

Source: Petrauskas (2006), Wirtgen (2007)

From the mechanistic analysis using KENLAYER, with all the input data mentioned before, the results are summarized in Table 10 and Figures 4 to 9.

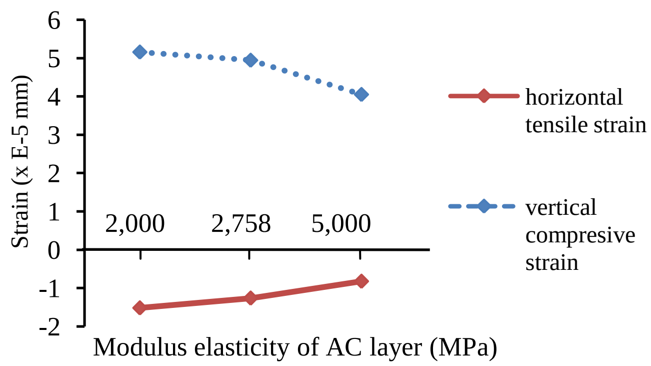

Figure 4. Effect of modulus elasticity of AC layer to the strain

Table 10. Result of sensitivity analysis in material properties variation

No			1	2	3	4
Layer	AC	CMR FB	AC	AC	CMR FB	CMR FB
Material	Asphaltic Concrete	Cold Milling Recycled Foam Bitumen	Asphaltic Concrete	Asphaltic Concrete	100% RAP + 1.5 – 2% Foamed Bitumen	100% RAP + 1.5 – 2% Foamed Bitumen
Variation of E (MPa)	2,758	1,241	5,000	2,000	2,500	1,000
ε_t	-1.26E-05		-8.22 E-06	-1.51 E-05	-1.12 E-05	-1.29 E-05
ε_c	4.95E-05		4.05 E-05	5.16 E-05	4.61 E-05	5.03 E-05
N_f	8.88E+08		2.00 E+09	6.04 E+08	1.34 E+09	8.08 E+08
N_d	2.58E+10		6.30 E+10	2.13 E+10	3.54 E+10	2.39 E+10
Design Life (year)	10		22.47	6.79	15.1	9.09

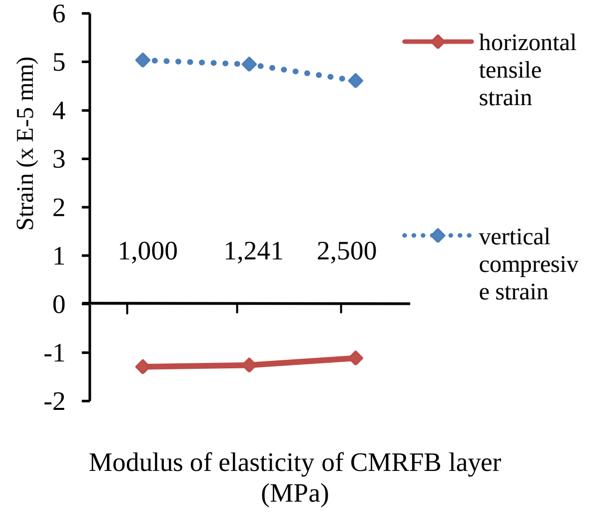

Figure 5. Effect of modulus elasticity of CMRFB layer to the strain

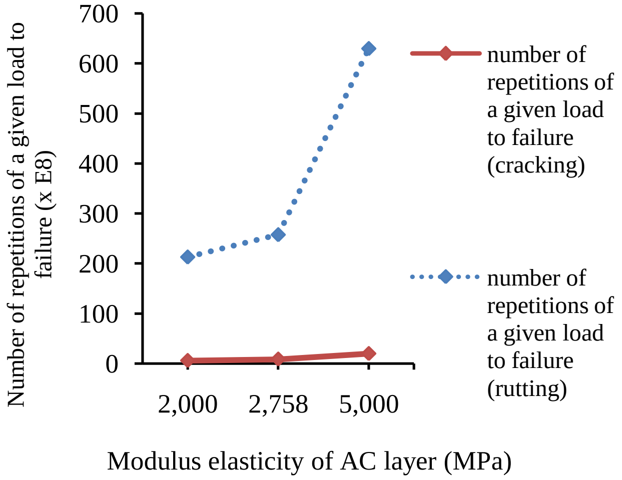

Figure 6. Effect of modulus elasticity of AC layer to the number of repetitions of a given load to failure

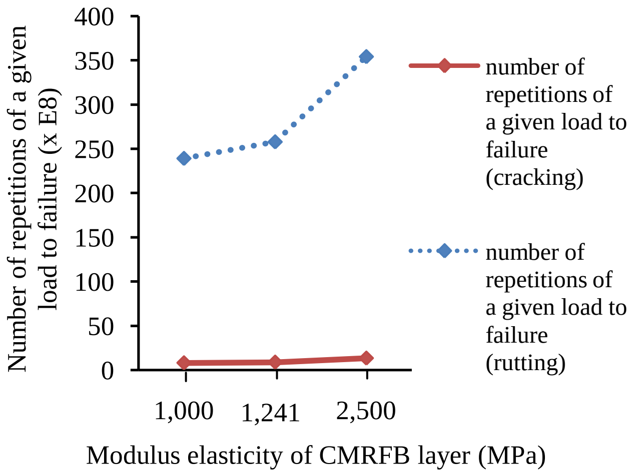

Figure 7. Effect of modulus elasticity of CMRFB layer to the number of repetitions of a given load to failure

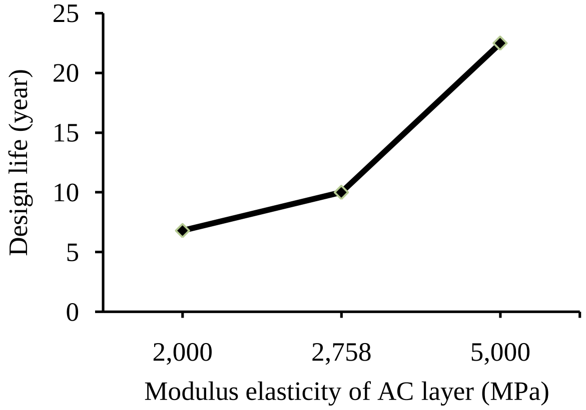

Figure 8. Effect of modulus elasticity of AC layer to the design life

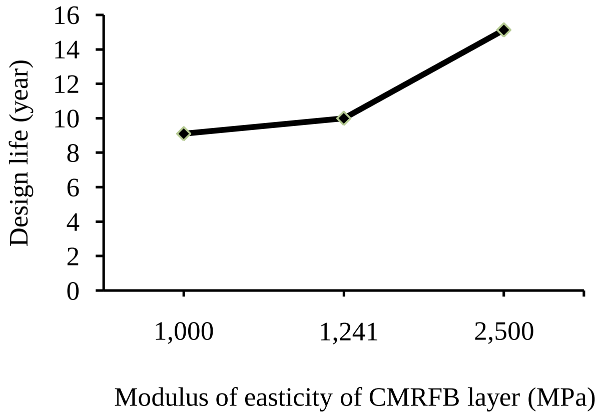

Figure 9. Effect of modulus elasticity of CMRFB layer to the design life

In case of variation in the modulus of elasticity of AC layer, if the modulus was decreased to 73%, the horizontal tensile strain and the vertical compressive strain increased by 120% and 104%, respectively. Moreover, the number of repetitions of a given load to failure for both rutting and cracking and the design life decreased by 68%, 83%, and 68%, respectively. On the other hand, if the modulus of elasticity of AC layer was increased by 181%, the horizontal tensile strain and the vertical compressive strain decreased by 65% and 82%, respectively. Moreover, the number of repetitions of a given load to failure for both rutting and cracking and the design life increased by 225%, 244%, and 225%, respectively.

Decreasing the modulus of elasticity of CMRFB layer to 81% had increased the horizontal tensile strain and the vertical compressive strain to 103% and 102%, respectively. Moreover, the number of repetitions of a given load to failure for both rutting and cracking and the design life decreased by 91%, 93%, and 91%, respectively. By contrast, if the modulus of elasticity of CMRFB layer was increased by 201%, the horizontal tensile strain and the vertical compressive strain became lower by 89% and 93%, respectively. The number of repetitions of a given load to failure for both rutting and cracking and the design life increased by 151%, 137%, and 151%, respectively.

D. Combination of Traffic Loading and Material Properties

In order to find the best solution to overcome the deterioration problem in flexible pavement design due to the overloading (150%, 200%, and 250% traffic loading), the combination of the upper bound of the modulus of elasticity of the two most top layers was analyzed using the KENLAYER. The input data used in LAYERINP is described in Table 11 and the result obtained from the KENLAYER is shown in Table 12.

Table 11. Scenario for combining traffic loading and material properties

No	Layer	Modulus of Elasticity (MPa)
1	AC	5,000
2	CMRFB	2,500
3	CTRB	220
4	Granular	70
5	Subgrade (embankment)	67
6	Ground	33

Table 12. Result of the combination

No	1	2	3
Traffic Loading (%)	150	200	250
ε_t	-7.55E-06		
ε_c	3.84E-05		
N_f	2.65E+09		
N_d	8.05E+10		
Initial Design Life (year)	1.61	0.46	0.18
Combined Design Life (year)	4.8	1.38	0.53

Based on Table 12, the increment of modulus elasticity increased the initial design life to around 300%. It was assumed that the increase of the modulus of elasticity of the two most top pavement layers had a great impact on the design life for the 150% traffic loading even though it still cannot restore the expected design life of 10 years. However, for the 200% and 250% traffic loading, these impacts appeared not as significant as the 150% traffic loading.

CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

The traffic load repetitions during the design period are very important to determine in flexible pavement design. The occurrence of 150%, 200%, and 250% overloading have a very significant effect in reducing 84%, 95%, and 98% of the pavement design life, respectively.

The variations in the modulus of elasticity of material affect the design life, even though not as significant as the traffic loading. The variation in asphaltic concrete layer has a higher effect on the design life than that on the cold milling recycling foam bitumen. If the modulus elasticity of asphaltic concrete decreases to the lower bound, the design life decreases by 32%, while if the modulus elasticity of cold milling recycling foam bitumen decreases to the lower bound, the design life decreases by only 9%. On the other hand, if the modulus elasticity of asphaltic concrete increases to the upper bound, the design life increases by 125%, while if the modulus elasticity of cold milling recycling foam bitumen increases to the upper bound, the design life increases by only 51%.

Both the horizontal tensile strain and the vertical compressive strain decrease if the modulus of elasticity of layer increases. Conversely, if the modulus of elasticity of layer decreases, the horizontal tensile strain and the vertical compressive strain increases.

The variations in the modulus of elasticity of each layer influence the number of repetitions of given load to failure. It affects the rutting failure more than the fatigue cracking failure.

The effect of overloading to the pavement structure performance is difficult to minimize even though the first two layers have relatively high modulus of elasticity. Compared to the value of modulus of elasticity of the asphaltic layer, the number of load repetitions affects more to the pavement design life. Because of that, for the purpose of increasing the pavement service life, it is more effective to limit and manage the allowable load than to improve the pavement materials.

B. Recommendations

The recommendations based on the result in this study are presented as follows:

1. In flexible pavement design, the consideration of using high quality of material properties is inappropriate if the load repetitions are still beyond the designer prediction.
2. The early deterioration in flexible pavement might be reduced if the overloading problems can be solved with the law enforcement.
3. The temperature change has an effect on the modulus of elasticity of asphaltic concrete layer. For further study, the impact of temperature change should also be accounted.
4. The overloading will decrease the traffic speed which in the end will also impact on the modulus of elasticity of asphaltic concrete layer. For further study, this phenomenon should also be accounted.

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