DEVELOPMENT OF MECHANISTIC DESIGN PROCEDURE OF FLEXIBLE PAVEMENT FOR TROPICAL CONDITION

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Abstract: Based on the analysis of several asphalt mixtures in Indonesia, generally, asphalt mixture has stiffness modulus \( E_1 \) varies from 2000 MPa to 5000 MPa. At the subgrade layer, soil has CBR value at 3% to 6% which is equivalent to modulus \( E_3 \) at 30 MPa to 60 MPa. The second layer would be a base layer with thickness considered to be a constant value at 200 mm and stiffness modulus \( E_2 \) at 100 MPa. The other characteristic is Poisson’s Ratio \((\nu)\). Poisson’s Ratio for surface \((\nu_1)\), base \((\nu_2)\) and subgrade \((\nu_3)\) was taken as 0.35; 0.40; and 0.45 respectively. Considering the material characteristic above then design nomograph were developed. The nomographs were inspired by Nothingham and there are then 4 types of nomographs related to a subgrade modulus. Pavement thickness result designed using mechanistic procedure was thicker than empirical method.

Key Words: Empirical, Mechanistic, Nomograph

1. INTRODUCTION

Flexible pavement design method for tropical country, like Indonesia, is generally adopted from other foreign country. This method was empirically developed based on many years experiences. By comparing the parameters such as environment, traffic loading and material characteristics, the condition will be significantly different and this will cause a different problem solving in design. Some methods adopted utilizing empirical data and try to build the correlation between inputs (such as loading, layer configuration, material, environment, etc.) and failure criteria based on experiences and experiments or both of them.

The promising method today is a mechanical method; structure phenomenon such as stresses, strains and deflections are described by the engineering and physical properties. Correlation between structures and physics are represented in mathematical model. In the application of mechanistic procedure, empirical elements used to determine the value of stress, strain and
deflection which are computed at the failure of the pavement. Physics phenomenon and failure parameters were used to develop the empirical equations to calculate the magnitude of loading until the pavement failure.

2. MODELLING PAVEMENT STRUCTURE

2.1 Assumptions

Flexible pavement was selected as the pavement type of study and was assumed to have three layer systems as surface layer (asphalt mix with aggregate), base layer (unbound granular material) and subgrade layer. Each layer has a different characteristics. The assumptions of pavement geometric are presented in Table 1 below.

<table>
<thead>
<tr>
<th>Layer composition</th>
<th>Kind of material</th>
<th>Required material properties</th>
<th>Assumption of material behavior</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st layer</td>
<td>Asphalt mixture</td>
<td>$E_1$, $\nu_1$</td>
<td>Elastic-linear</td>
<td>Questioned</td>
</tr>
<tr>
<td>2nd layer</td>
<td>Unbound granular material</td>
<td>$E_2$, $\nu_2$</td>
<td>Elastic-linear</td>
<td>Constant (200 mm)</td>
</tr>
<tr>
<td>3rd layer</td>
<td>Subgrade</td>
<td>$E_3$, $\nu_3$</td>
<td>Elastic-linear</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes: $E =$ Stiffness modulus (MPa)  
$\nu =$ Poisson’s Ratio

Loading type was assumed to have three types which are single axle, tandem axle and three axles. The loading are easy to found in the national roadway. Wheel configuration for each type of loading are illustrated and shown on Figures 1 and 2.

2.2 Stiffness Modulus of Asphalt Mixture

In order to provide a sufficient result, modulus values were determined from material which are widely used in Indonesia. Experiments have been done for all mixtures normally used in Indonesia such as Asphalt Concrete-Wearing Course (AC-WC), Asphalt Concrete – Binder Course (AC-BC), Stone Mastic Asphalt and other mixtures containing some additives.

In the analysis of the modulus ($E_1$), there are several point could be informed about asphalt mixtures in Indonesia:

- At the same time of loading, increasing the field temperature from 30°C up to 35°C will significantly influence the modulus value. When the field temperature increases at 5°C, modulus will down up to 10%.

- The other condition, at the same field temperature, increasing time of loading (design speed change from 50 kph to 60 kph, +20%) has not significantly affect the modulus. The speed will raise the modulus only for 0.38%.

- Overall case, stiffness modulus varies from 394 MPa up to 7153 MPa. The modulus distribution for all combination of temperature and time of loading are presented in Table 2.
From the above table, most of $E_1$ value was varies from 2000 MPa up to 5000 MPa. This result allows the development of design nomographs for many type of $E_1$. In this investigation, modulus were assumed and considered to be $E_1 = 1000$ MPa, $E_1 = 2000$ MPa, $E_1 = 3000$ MPa, $E_1 = 4000$ MPa dan $E_1 = 5000$ MPa.

Table 2 Classification of $E_1$ values

<table>
<thead>
<tr>
<th>Range of $E_1$ (MPa)</th>
<th>Condition of analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_1 \leq 1000$</td>
<td>S = 50 kph T = 30°C</td>
</tr>
<tr>
<td>1000 &lt; $E_1 \leq 2000$</td>
<td>18 9 7 6</td>
</tr>
<tr>
<td>2000 &lt; $E_1 \leq 3000$</td>
<td>21 49 20 57</td>
</tr>
<tr>
<td>3000 &lt; $E_1 \leq 4000$</td>
<td>46 27 50 30</td>
</tr>
<tr>
<td>4000 &lt; $E_1 \leq 5000$</td>
<td>26 17 22 18</td>
</tr>
<tr>
<td>5000 &lt; $E_1 \leq 6000$</td>
<td>14 8 16 4</td>
</tr>
<tr>
<td>6000 &lt; $E_1 \leq 7000$</td>
<td>5 6 2 0</td>
</tr>
<tr>
<td>$E_1 &gt; 7000$</td>
<td>0 1 0 0</td>
</tr>
</tbody>
</table>

2.3 Kenpave’s Input Data
Considering the analysis to material characteristics mentioned above, the input parameter for Kenlayer program could be then assumed as follows.

Table 3 Values of Modulus ($E_i$), Poisson’s Ratio ($\nu_i$) and layer thickness ($H_i$)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_1$</td>
<td>Varied: 1000, 2000, ..., 5000 MPa</td>
<td></td>
</tr>
<tr>
<td>$E_2$</td>
<td>Constant: 100 MPa</td>
<td></td>
</tr>
<tr>
<td>$E_3$</td>
<td>Varied: 30, 40, 50, 60 MPa Equivalent to CBR value at 3, 4, 5, 6%</td>
<td></td>
</tr>
<tr>
<td>$\nu_1$</td>
<td>Constant: 0.35</td>
<td></td>
</tr>
<tr>
<td>$\nu_2$</td>
<td>Constant: 0.40</td>
<td></td>
</tr>
<tr>
<td>$\nu_3$</td>
<td>Constant: 0.45</td>
<td></td>
</tr>
<tr>
<td>$H_1$</td>
<td>Varied: 50, 60, ..., 150 mm</td>
<td></td>
</tr>
<tr>
<td>$H_2$</td>
<td>Constant: 200 mm</td>
<td></td>
</tr>
</tbody>
</table>

2.4 Calculation of Stress and Strain
In the fatigue analysis, the horizontal minor principal strain, instead of the overall minor principal strain, was used. The strain was called minor because tensile strain is considered negative. Horizontal principal strain is used because it is the strain that causes the crack to initiate at the bottom of asphalt layer. The horizontal principal tensile strain is determined from:

$$\varepsilon_i = \frac{\varepsilon_x + \varepsilon_y}{2} - \sqrt{\left(\frac{\varepsilon_x - \varepsilon_y}{2}\right)^2 + \gamma_{xy}^2}$$

(1)

Where:

- $\varepsilon_i$ = principal horizontal tensile strain at the bottom of asphalt layer
- $\varepsilon_x$ = strain in the x direction,
- $\varepsilon_y$ = strain in the y direction,
- $\gamma_{xy}$ = the shear strain on the x plane in the y direction, and
\[ \varepsilon_x = \frac{1}{E} \left[ \sigma_x - v(\sigma_y + \sigma_z) \right] \]  \hfill (2)

\[ \varepsilon_y = \frac{1}{E} \left[ \sigma_y - v(\sigma_x + \sigma_z) \right] \]  \hfill (3)

\[ \gamma_{xy} = \frac{2(1 + v)}{E} \tau_{xy} \]  \hfill (4)

Damage analysis is performed for both fatigue cracking and permanent deformation.

\[ N_f = f_1(\varepsilon_t)^{-f_2}(E_1)^{-f_3} \]  \hfill (5)

Where:
- \( N_f \) = the allowable number of load repetitions to prevent fatigue cracking,
- \( \varepsilon_t \) = the tensile strain at the bottom of asphalt layer,
- \( E_1 \) = elastic modulus of asphalt layer,
- \( f_1, f_2, f_3 \) = constants, determined from laboratory fatigue test
  - \( 0.00796; 3.291 \) and \( 0.854 \) (Asphalt Institute)
  - \( 0.0685; 5.671 \) and \( 2.363 \) (Shell)

The failure criterion for permanent deformation is expressed as

\[ N_d = f_4(\varepsilon_c)^{-f_5} \]  \hfill (6)

Where:
- \( N_d \) = the allowable number of load repetitions to limit permanent deformation,
- \( \varepsilon_c \) = the compressive strain on the top of subgrade
- \( f_4, f_5 \) = constants determined from road test or field performance
  - \( 1.365 \times 10^{-9} \) and \( 4.477 \) (Asphalt Institute)
  - \( 6.15 \times 10^{-7} \) and \( 4.0 \) (Shell)
  - \( 1.13 \times 10^{-6} \) and \( 3.571 \) (University of Nottingham)

2.4 Development of Design Nomographs
Strain analysis has been done at two locations, below asphalt layer (\( \varepsilon_t \)) and above subgrade layer (\( \varepsilon_c \)). The analysis was done with strain concept as mentioned in Kenpave Software. Composition of traffic loading was taken from traffic survey at Java Island roadway network. With 4 variations of subgrade modulus (\( E_3 = 30, 40, 50 \) and \( 60 \) MPa), four nomographs were then resulted, one of them is shown in Figure 3 as a sample.

3. IMPLEMENTATION

3.1 Input Data

3.1.1 Traffic Condition
Traffic condition data was generated from Weigh in Motion (WIM) survey at Jakarta-Cikampek Toll Road (see Table 4). Vehicle was classified in to 13 class, otherwise there are several types which are not found in Indonesia.
Table 4 Data of vehicle types in each direction

<table>
<thead>
<tr>
<th>Vehicle Class</th>
<th>Number of Vehicle (veh)</th>
<th>Average Vehicle Weight (kg)</th>
<th>Number of Vehicle (veh)</th>
<th>Average Vehicle Weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10,420</td>
<td>1,857</td>
<td>8,335</td>
<td>2,045</td>
</tr>
<tr>
<td>2</td>
<td>10,564</td>
<td>7,318</td>
<td>11,373</td>
<td>9,172</td>
</tr>
<tr>
<td>12</td>
<td>1,076</td>
<td>11,731</td>
<td>1,002</td>
<td>9,049</td>
</tr>
<tr>
<td>3</td>
<td>4,666</td>
<td>23,269</td>
<td>4,488</td>
<td>25,426</td>
</tr>
<tr>
<td>5</td>
<td>1,722</td>
<td>25,992</td>
<td>1,789</td>
<td>26,483</td>
</tr>
<tr>
<td>8</td>
<td>1,343</td>
<td>29,215</td>
<td>893</td>
<td>28,554</td>
</tr>
<tr>
<td>9</td>
<td>202</td>
<td>34,099</td>
<td>346</td>
<td>36,582</td>
</tr>
<tr>
<td>10</td>
<td>515</td>
<td>33,102</td>
<td>481</td>
<td>33,806</td>
</tr>
<tr>
<td>11</td>
<td>583</td>
<td>41,256</td>
<td>597</td>
<td>37,274</td>
</tr>
</tbody>
</table>

3.1.2 Asphalt Mixture
The proposed roadway assumed to use AC-WC. This type of mixture has a voids in mineral aggregate (VMA) of 15.2%. Meanwhile, the asphalt itself has a penetration and softening point of 64 and 51°C respectively.

3.1.3 Additional Data
From the data surveyed in related location, field temperature was estimated to be 35°C and the vehicle design speed was 60 kph. Soil investigation results show that soil in the area has 5% CBR value. In order to predict the future traffic, 5% growth rate per annum was assumed. Using all data mentioned above, estimation of AC-WC is required handle the traffic for 5 years since the first road opening.

3.2 Designing the Pavement Thickness

3.2.1 Determination of Asphalt Mixture Stiffness Modulus ($E_1$)
Modulus was computed as a function of asphalt modulus ($S_{bit}$) and VMA. VMA was defined above and $S_{bit}$ could be determined using the Van der Poel nomograph (as shown on Figure 4). On that nomograph, for design speed at 60 kph with pavement temperature at 35°C, make a vertical line from “time of loading” at 1/60 = 0.016 and will cross the “temperature difference” at (64-35)°C = 29°C. Next step is to calculate the Penetration Index (PI) with the following equation:

$$PI = \frac{(20 - 500 \cdot A)}{(1 + 50 \cdot A)}$$

$$A = \frac{(Log800 - Log\cdot Pen)}{(TL - 25)}$$

(7)

For asphalt with properties calculated above the PI is (-) 0.35. With PI value of (-) 0.35, drawn the horizontal line from PI axis to the right until find the last crossing then draw a curve line to $S_{bit}$ axis. Substitute the $S_{bit}$ and VMA to equation.

$$E_1 = S_{bit} \left(1 + \frac{257.4 - 2.5VMA}{n(VMA - 3)}\right)^n$$

(8)

where $n = 0.83 \cdot Log\left(\frac{4 \cdot 10^{10}}{S_{bit}}\right)$. The stiffness modulus of mixture is 5,404 MPa.
3.2.2 Computation of Traffic Design During Service Life
The calculation of Cumulative ESAL (CESAL) was done using the common formula, that
ESAL is a function of \( \left( \frac{W}{8160} \right)^4 \), where W is axle weight. For different axle configuration,
this equation needs to be modified with additional constant in front of the equation. The
average CESAL for Cikampek direction was 18,219.4 per day and for direction to Jakarta was
21,961.9 per day. Using the highest value as initial traffic with 5% growth rate, for 5 next
years there would be 21,961.9 x 365 (1+5%)^5 = 10.2 million ESAL.

3.2.3 Determination of Subgrade Modulus (E_3)
Subgrade modulus was determined approximately 10 x CBR (MPa). With the CBR value at
5% then the modulus should be 50 MPa. This computation is a simple one, in the field there
must be a representative CBR value for many tested point.

3.2.4 Determination of Tensile Strain (\( \varepsilon_c \)) and Compressive Strain (\( \varepsilon_t \))
In the mechanistic procedure, fatigue crack and permanent deformation were used to measure
the time when the pavement is failed. Equation as a correlation between traffic loading, \( E_1 \)
and strain are mentioned in equation (8) and (9). Table 5 presents the input parameter and the
strain which is questioned.

<table>
<thead>
<tr>
<th>Mixture type</th>
<th>N</th>
<th>( E_1 )</th>
<th>( f_1 )</th>
<th>( f_2 )</th>
<th>( f_3 )</th>
<th>( f_4 )</th>
<th>( f_5 )</th>
<th>( \varepsilon_c )</th>
<th>( \varepsilon_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC-WC</td>
<td>10,230,792</td>
<td>5,404</td>
<td>0.0685</td>
<td>5.671</td>
<td>2.363</td>
<td>6.15 x 10^{-7}</td>
<td>4.0</td>
<td>4.95 x 10^{-4}</td>
<td>10.07 x 10^{-4}</td>
</tr>
</tbody>
</table>

Note: in this strain calculation, constants from Shell are used to determine the strain.

3.2.5 Determination of Asphalt Thickness
Asphalt thickness according to fatigue cracking and permanent deformation criteria was
determined by using the appropriate nomograph, there is a nomograph of \( E_3 = 50 \) MPa. By
plotting the strain to the nomograph results asphalt thickness required. Fatigue cracking and
permanent deformation criteria require 79 mm and 50 mm asphalt thickness respectively and
the result was rounded up into 80 mm as the adopted design.

<table>
<thead>
<tr>
<th>Mixture type</th>
<th>Thickness according to (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fatigue Cracking</td>
</tr>
<tr>
<td>AC-WC</td>
<td>79</td>
</tr>
</tbody>
</table>

3.3 Mechanistic versus Empirical Methods
After the finishing of mechanistic procedure, it is necessary to compare the results between
mechanistic and empirical methods. The determination of pavement thickness on empirical
method was based on the relative strength for each layer using the following equation:

\[ ITP = a_1D_1 + a_2D_2 \] (9)

Where:
- ITP = pavement thickness index
- \( a_1, a_2 = \) relative strength coefficient
D_1, D_2 = thickness for each layer (cm)

ITP is also could be calculated by using this equation:

\[ \text{Log} (\text{LER} \times 3560) = 9.36 \left( \frac{\text{ITP}}{2.54} + 1 \right) - 0.20 + \frac{\text{Log} \left( \frac{\text{IP}_0 - \text{IP}_T}{4.2 - 1.5} \right)}{1904} + \frac{\text{ITP}}{0.40 + \left( \frac{1}{\text{FR}} \right)^{5.19}} + 0.372 (\text{DDT} - 3) \] (10)

Where:
LER = cumulative of standard axle during service life
IP_0 - IP_T = the difference between initial design serviceability and design terminal serviceability indexes
DDT = soil support value
FR = regional factor

To compare the results of empirical and mechanistic procedure, thickness of D_2 in empirical method was assumed to be same with H_2 in mechanistic, 200 mm. Thickness as a result of empirical method, the result is generally higher than the mechanistic one. From 80 random sample, using variations of E_1, E_3 and N, the results show that the average thickness according to empirical method is 23.3 cm. Meanwhile the mechanistic requires only 7.8 cm. According to the theory, it is logically right since the mechanistic procedure is depended on the failure criteria, otherwise the empirical was based on the experiences in other places.

The thickness difference for those two methods is 15.4 cm for overall sample quoted. This difference occurred because when modeling the pavement using empirical method, D_3 was assumed to be 0 (zero), resulted in higher D_1 value. Anyway, in whatever method selected, if the quality is well controlled, the pavement should be in the safe side, but for the same condition of mechanistic procedure, efficiency in budget will be obtained.

4. DEVELOPMENT OF COMPUTER PROGRAM

In the fast growing information technology era, almost all computing could be done by utilizing a computer program. The computer will reduce the human error especially when plotting some information in the charts. Therefore, the study also develops the flowchart of mechanistic computer program. Nomograph designs, which are mentioned above were expected to cover all material properties available and normally used in Indonesia as a prototype of tropical area and may be used to determine the pavement thickness. Thickness generated from the nomograph is much depended on the skill of the designer when plotting the information to the charts. When the exact number is required to minimize cost (but with the same specification), it is become important to change the nomograph to the mathematical equations. This equation could be then translated to the computer language in the program.

To develop the computer program, the data required consists of characteristic of asphalt, characteristic of mixtures, soil support values, traffic loading, pavement temperature and design speed. Asphalt characteristic needed are penetration and softening point values. VMA is then used to represent mixture characteristics. The main flowchart of the computer program is described in Figure 5.
5. CONCLUSIONS

1. The Nomograph of mechanistic procedure has been prepared for 4 (four) values of Subgrade Modulus (E<sub>3</sub>) and each nomograph considered 5 (five) values of Asphalt Mixture Modulus (E<sub>1</sub>).

2. The asphalt mixture modulus (E<sub>1</sub>) has some characteristics as follows:
   a. When the value of E<sub>1</sub> was less than 1000 MPa, then the thickness according to permanent deformation could not be determined.
   b. When the value of E<sub>1</sub> was too high (≥4000 MPa) and the number of cumulative standard axle higher than 7 million, then give the thickness below 50 mm.
   c. When the value of E<sub>1</sub> was 2000 MPa, then the thickness of asphalt layer was depended on the permanent deformation criteria.
   d. When the value of E<sub>1</sub> was higher than 3000 MPa, then the thickness was depended on the fatigue cracking criteria.

3. Comparing the mechanistic procedure and the empirical procedure, the first one require less asphalt thickness. This is due to the assumptions that the subbase thickness was equal to zero.

4. The Nomograph of mechanistic procedure should be developed continuously and updated with some additional data, such as the data of various specific material obtained in the different region in Indonesia.

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Yang H.Huang, (1993) **Pavement and Analysis Design.** USA, Prentice Hall.

![Figure 1 Single-axle and tandem-axle loading configurations](image1)

![Figure 2 Three-axle loading configuration](image2)
Figure 3 Nomograph of asphalt thickness determination for Subgrade Modulus ($E_3$) = 50 MPa
($E_2$ = 100 MPa and $H_2$ = 200 mm)
Figure 4 Van der Poel’s Nomograph
Figure 5 Flow Chart of pavement thickness calculation