ASSESSMENT OF HOURLY VARIATION IN STRUCTURAL CAPACITY OF A PAVEMENT BASED ON BACK CALCULATION PROCEDURE

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Abstract: To address problems due to continuous variations in pavement strength, a research had been conducted to analyze deflection data, measured on 3 different points along a one-km road section 4 times a day with a 6-hour interval by using FWD at 3 different loadings. Accurate layer thickness was obtained from a core test. The result showed that applying a temperature correction factor on deflection data was found to be ineffective; it also suggested to modify the existing model for estimating asphalt layer modulus to account for stresses occurring therein; a weak bonding layer between the asphalt and the granular layers was further observed and this conforms with the previous study (Hatchiya, 1998); then, a novice model for estimating structural capacity of a pavement from back calculated modulus was proposed. Finally, this research suggests that the analysis presented should be adopted in practice for detailed investigation survey prior to any major pavement rehabilitation work.

Key Words: temperature correction, modulus, back calculation, pavement structural capacity.

1. INTRODUCTION

Back calculation to determine pavement layer modulus from field deflection data is accepted at present as the best technique available for structural evaluation of an existing pavement. The resulting pavement layer modulus is useful to identify quantitatively any weakness found in the existing pavement, from which proper maintenance activities may be determined more accurately and objectively.

One major problem faced with this promising technique is that deflection data, and hence pavement strength, varies from time to time due to changes in pavement temperature and the season, apart from essentially being caused by the cumulative axle loading. On the other hand, deflection data measurement is only carried out at a certain time once a year, particularly, when a pavement has experienced signs of distresses. In the past, the effects of temperature and the season on pavement deflection are taken into account by applying a correction factor only to the maximum deflection value. However, this approach has been proven ineffective since deflection bowl profile that reflects the vertical distribution of layer modulus in the pavement is also influenced by these two factors (Thomson, 1991).

A research had been conducted to analyze pavement surface deflection data that was measured on 3 different points along a one-km road section 4 times a day with a 6-hour interval by using Falling Weight Deflectometer. The loadings applied were 30, 40 and 50 kN in order to assess whether layer modulus is a function of stresses occurring in the pavement. Afterwards, coring was also made on each measurement point to obtain accurate pavement layer thickness data.
With the current advances in computerized analytical design tools, detailed variations in pavement conditions can be analyzed quite conveniently. Unfortunately, the resulting back calculated modulus has not yet been employed to estimate the structural capacity of an existing pavement by using this analytical design tools (AASHTO, 1993). This paper is intended primarily to outline the use of an analytical design tools in assessing the structural capacity of a pavement.

2. BACK CALCULATION ALGORITHM

Computer program BackCalc is used here to analyze pavement surface deflection data. The main algorithms should be employed for back calculation processes have been described in the previous paper (Kosasih, et.al., 2003). It can be argued that back calculation will be best suited for analyzing a pavement structure modeled as a four-layered system consisting of two asphalt layers, one granular layer and the subgrade. However, this model may sometimes result in a rather illogical pavement layer modulus, such as the modulus of granular layer is smaller than that of the subgrade, or a very high modulus of asphalt layer.

For this research, a great deal of efforts has been spent to overcome such a problem. Lack of bonding between asphalt layer and granular layer as addressed by Hatchiya, et.al. (1998) is apparently one of the factors that may cause illogical pavement layer modulus resulting from back calculation. A bonding layer is then suggested to be incorporated in modeling a pavement structure as a four-layered system by replacing one of the two asphalt layers.

If there is no bonding problem between asphalt layer and granular layer, the modulus of the bonding layer will be similar to the asphalt layer or the granular layer. On the other hand, if bonding problem does exist, the modulus of the bonding layer will be very small.

This finding improves the modular ratio criterion that should be used for back calculation processes, as follows:

\[
\frac{E_{\text{grain layer}}}{E_{\text{subgrade}}} \geq 1.0 \pm 5.0 \quad \ldots (1a)
\]

\[
\frac{E_{\text{asphalt layer}}}{E_{\text{bonding layer}}} \geq 1.0 \quad \ldots (1b)
\]

3. DATA

3.1. Pavement Structure Data

Pavement structure data used for analysis was obtained from a core test conducted on the three deflection measurement points (STA 0+000, 0+500, 1+000) and from the technical design document. The existing pavement has 5 distinctive layers comprising two asphalt layers, two unbound granular base layer and subbase layer, and cohesive soil subgrade. As mentioned earlier, this pavement is then modeled as a four-layered system structure including that of the bonding layer between the asphalt layer and the unbound granular base layer, as shown in Figure 1.

As shown in the figure, asphalt layer thickness of the three deflection measurement points varies slightly. For this paper, the bonding layer thickness is fixed at 1 mm which forms a
part of the granular layer. A more detailed study is still progressing to assess the best representation of the bonding layer. The significance of this bonding layer in back calculation process will be described later with an illustrative example.

The poisson ratios of the asphalt layer, granular layer and cohesive soil are assumed to be 0.4, 0.3 and 0.4 respectively.

3.2. Deflection Data

Variations in deflection data ($d_{\text{max}}$) due to variations in pavement temperature, FWD test load and the test site are shown in Figure 2a.

The effect of pavement temperature on deflection data for the three test sites appears to be different, but it is definitely not linear. Instead, it forms a loop as clearly indicated by the data from STA 1+000. The first deflection data was measured at 06:00am with pavement temperature of 24°C. The measurement was then repeated at 12:00pm, 18:00pm and 00:00am with pavement temperature of 45°C, 32°C and 27°C respectively. This data also suggests that the maximum pavement temperature may still even be higher, and this probably occurs at around 14:00pm. Therefore, more readings should reasonably be added if a similar test is repeated again in the future.

A non-linear effect of pavement temperature on deflection data indicates that there is a temperature gradient within the asphalt layer in the morning and in the evening. In the morning, the upper asphalt layer is hotter. Meanwhile, in the evening, the lower asphalt layer is warmer. This situation proves implicitly that temperature correction factor for deflection data will never be unique. Figure 2b shows an example how pavement temperature affects deflection bowl profile non-linearly for the pavement at STA 0+000.

As expected, pavement deflection will be higher, if the test load increases. Later, it will be shown that if the test load increases, asphalt layer modulus will also be higher. This stress dependent asphalt layer modulus is of quite importance in evaluating structural capacity of a
pavement. At present, pavement deflection at one test site is measured three times with the same test load. This research suggests that deflection readings should preferably be recorded six times for three different test loads, i.e., each test load is applied twice. This then enables a wider load spectrum, not just the standard axle, be analyzed proportionally by using Miner’s Law (Asphalt Institute, 1982).

Comparing deflection data from the three test sites, one may decide, traditionally, that the pavement structure at STA 1+000 is the weakest because it has the highest $d_{\text{max}}$. In most cases, deflection bowl data is rarely taken into consideration. Later, it will be shown that the weakest pavement structure is actually occurred at STA 0+000 triggered by the weakest asphalt layer modulus as resulting from back calculation.

Figure 2a. Deflection data ($d_{\text{max}}$) measured at different pavement temperatures, test loads and the test sites

Figure 2b. Deflection bowl data of STA 0+000 measured at different pavement temperatures and test loads
4. PAVEMENT TEMPERATURE CORRECTION FACTOR

In analyzing pavement deflection data, temperature correction factor \((f_t)\) and seasonal correction factor \((f_s)\) are traditionally needed to calculate standard deflection at a specified standard temperature and standard wet season. But, it has been argued previously that this approach may be incorrect because of temperature gradient variations experienced in the asphalt layer during the day.

Figure 3 confirms that argument. In the figure are plotted two temperature correction factor curves according to NAASRA (1987) for asphalt layer thicknesses, \(D\), of 100 and 150 mm respectively. Also plotted are the temperature correction factors calculated from the deflection data presented in Figure 2a, where the asphalt layer thickness, \(D \approx 110\) mm, with the reference temperatures of 32°C and 45°C.

The observed temperature correction factors are obviously too scattered; and they can hardly be used to defining a unique temperature correction factor curve.

Therefore, deflection data is essentially not required to be corrected neither for pavement temperature nor for the season. Instead, the resulting back calculated pavement modulus needs to be expanded to represent various pavement conditions possible. For tropical countries like Indonesia, 12 pavement conditions subjected to 4 pavement temperatures in a day and 3 weathers in a year may be sufficient. All these pavement conditions are then analysed proportionally to assess the overall structural capacity of the pavement.

5. BACK CALCULATED MODULUS

Back calculated modulus as shown in Figure 4 is obtained from the deflection data and the pavement structure data presented above by using program BackCalc. In line with the previous discussion, no temperature correction is applied here to the deflection data. The figure shows the effects of pavement temperature, test load and the test site on the modulus of asphalt layer, granular layer and the subgrade.

The effect of pavement temperature on the asphalt layer modulus unfortunately rather varies. For the pavement at STA 0+000 and 0+500, the effect is roughly linear. Meanwhile, for the pavement at STA 1+000, a non-linear effect is observed. However, more data is apparently still required to explain such a variation.

Pavement temperature also affects the granular layer modulus to a lesser extent. This effect may be related to the changes in stress distribution within the asphalt and granular layers.
Surprisingly, for this pavement, the modulus of the subgrade is more or less constant. This finding proves the fact that the asphalt and the granular layers usually carry most of the stresses induced by a wheel load.

Unexpectedly, test load also affects the asphalt layer modulus significantly. The higher the test load, the higher will be the asphalt layer modulus. This is contradictory to the existing theoretical models that stress level in an asphalt layer do not affect its modulus. A typical theoretical model is shown in Equation (2).

Unexpected result is further shown regarding the relationship between maximum deflection and asphalt layer modulus. With reference to Figure 2a, a smaller maximum deflection does not always produce a higher asphalt layer modulus as demonstrated by the pavement at STA 0+000. Yet, its subgrade modulus is the highest. It will be shown later that this pavement is actually found to be the weakest. This unexpected results suggest potential advantages of using back calculation process for pavement evaluation purposes, since it can identify unseen problems within a pavement structure.

All these pavement modulus values are then analysed proportionally to assess the overall structural capacity of the pavement.

The modulus of the bonding layer between asphalt layer and granular layer is not shown in Figure 4. This is because its value is either to small (lack of bonding) or in between the asphalt layer modulus and the granular layer modulus (good bonding). Figure 5 illustrates the significance of bonding layer in back calculation process. In Figure 5a, without bonding
layer, the modulus of granular layer appears to be unreasonable as it is smaller than the subgrade modulus. This problem can then be alleviated by the presence of a bonding layer as shown in Figure 5b.

Figure 5a. Back calculated modulus for a pavement modeled as a three-layered system (without bonding layer)

Figure 5b. Back calculated modulus for a pavement modeled as a four-layered system (with a bonding layer)

6. THEORETICAL MODEL FOR ASPHALT LAYER MODULUS

A typical theoretical model for estimating asphalt layer modulus is proposed by the Asphalt Institute (1982), as follow:

$$E_p = 100,000 \times (10)^{0.1}$$

where: $\beta_1 = \beta_2 + 0.000005 \beta_2 - 0.00189 \beta_2 / f^{0.1}$

$\beta_2 = (0.483 V_d)^{0.5} \left(T^{1.3}+0.49823 \log \theta \right)$

$\beta_3 = 0.55383 + 0.028829 E_{200} / f^{0.17033} - 0.03476 V_r + 0.070377 \eta$

$+ 0.931757 / f^{0.02774}$

$E_p =$ asphalt layer modulus, psi

$V_r =$ voids in mix, %

$V_d =$ volume of bitumen, %

$E_{200} =$ aggregate portion passing #200 sieve, %

$f =$ loading frequency, Hz

$T =$ pavement temperature, °F

$\eta =$ asphalt viscosity at 70°F, 10^6 poises
Mix characteristics required as input to Equation (2) must be obtained from laboratory tests. Typical values used for this research are as follow:

\[
\begin{align*}
\rho_t & = 4 \, (\%) \\
\rho_b & = 12 \, (\%) \\
P_{500} & = 5 \, (\%)
\end{align*}
\]

Table 1 shows the asphalt layer moduli calculated by using Equation (2), together with those resulting from back calculation for test load of 40 kN at four pavement temperatures measured at the time of deflection survey. In general, the theoretical modulus and the back calculated modulus are different. There are three interesting points to be discussed in more detail here:

a. At 45°C, theoretical modulus is smaller than back calculated modulus. This is consistent with the theory stated by Shell (1978) that in visco-elastic conditions the modulus of an asphalt mix is influenced not only by the stiffness of asphalt but also by the shape, the gradation and the interlocking state of the aggregates.

b. The difference found between theoretical modulus and back calculated modulus indicates that asphalt layer modulus on site decreases continuously by time or by wheel loads. This suggests that, time-series analysis is of importance to be performed in pavement evaluation works.

c. Asphalt layer modulus, like subgrade modulus and granular layer modulus, is stress-dependent.

In brief, back calculated modulus of asphalt layer after being corrected by the stress level occurring therein should be used in evaluating the structural capacity of a pavement, and further used in the overlay design process.

Table 1: Comparison between theoretical modulus and back calculated modulus

<table>
<thead>
<tr>
<th>Pavement Temperature (°C)</th>
<th>Representative Time Range</th>
<th>Theoretical Modulus (MPa)</th>
<th>Back Calculated Modulus (MPa), at STA:</th>
<th>0+000</th>
<th>0+500</th>
<th>1+000</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 08:00</td>
<td>4,570.01</td>
<td>1,863.87</td>
<td>2,436.04</td>
<td>3,234.38</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>10:00 - 16:00</td>
<td>595.41</td>
<td>1,262.07</td>
<td>2,179.92</td>
<td>1,631.23</td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>08:00 - 10:00</td>
<td>2,259.38</td>
<td>1,577.76</td>
<td>2,307.86</td>
<td>1,748.56</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 02:00</td>
<td>3,548.86</td>
<td>1,506.59</td>
<td>2,238.21</td>
<td>2,616.66</td>
<td></td>
</tr>
</tbody>
</table>

7. DESIGN CRITERIA

Design criteria are required for both evaluating the structural capacity of a pavement and for designing overlay thickness. For this research, four design criteria are proposed:

a. Maximum deflection criterion; from past studies (Kennedy, et.al., 1978 dan NAASRA, 1987), it is known that maximum deflection criterion is applicable only for a pavement that will fail due to rutting. Such a pavement normally has a thick asphalt layer and a thin aggregate layer. The relationship between cumulative number of standard axles, \(N_{d_{\text{max}}}\) (ESA) and maximum deflection, \(d_{\text{max}}\) (0.001 mm) is:

\[
d_{\text{max}} = \frac{\log \left( N_{d_{\text{max}}} \right)}{-9.98131 + 2.54999 \log \left( N_{d_{\text{max}}} \right)} \quad \ldots (2a)
\]
b. Curvature criterion; according to NAASRA (1987), curvature of a deflection bowl is defined as a difference between maximum deflection, \( d_{\text{max}} \) (0.001 mm), and offset deflection, \( d_{200} \) (0.001 mm), at 200 mm apart from the measured point. This criterion is normally related to fatigue cracking as failure mode.

\[
(d_{\text{max}} - d_{200}) = 4.05498 \left( N_{\text{def}} \right)^{-0.2297} 
\]  

\( \ldots (2b) \)

c. Rutting criterion; rutting criterion according to Asphalt Institute (1982) is determined only by vertical compressive strain occurring on top of the subgrade (\( \varepsilon_c \)).

\[
N_{\text{def}} = 0.001365 \varepsilon_c^{-4.471} 
\]  

\( \ldots (2c) \)

d. Fatigue cracking criterion; according to Asphalt Institute (1982), fatigue cracking is not only determined by horizontal tensile strain underneath an asphalt layer (\( \varepsilon_t \)), but also by mix characteristics, including voids in mix, \( V_v \) (%), volume of asphalt, \( V_b \) (%), and asphalt layer modulus, \( E_p \) (MPa).

\[
N_{\text{retak}} = 18.4 \left( 10^{4.84 \left( \frac{V_v}{V_b + V_v} - 0.69 \right)} - 0.004325 \varepsilon_t^{-3.291} \right) \left( 144.8582 E_p^{-0.854} \right) \]  

\( \ldots (2d) \)

The values of \( N_{\text{dmax}}, N_{d200}, N_{\text{def}}, N_{\text{retak}} \) represent the structural capacity of an existing pavement based upon each design criterion. The smallest is normally reported as the remaining life of the pavement.

Table 2: Calculation of the remaining life of the pavement at STA 0+000

(a) Maximum Deflection Criterion

<table>
<thead>
<tr>
<th>Pavement Temp (°C)</th>
<th>Representative Time Range</th>
<th>FWD Deflection (0.001 mm)</th>
<th>BB Deflection (0.001 mm)</th>
<th>( N_{\text{dmax}} ) (SS)</th>
<th>Traffic Loading (%)</th>
<th>Remaining Life (SS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 06:00</td>
<td>623</td>
<td>388.44</td>
<td>10.903</td>
<td></td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>10:00 - 16:00</td>
<td>579</td>
<td>402.72</td>
<td>44.997</td>
<td></td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>16:00 - 20:00</td>
<td>576</td>
<td>416.67</td>
<td>39.549</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 02:00</td>
<td>576</td>
<td>416.76</td>
<td>5.591</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(b) Curvature Criterion

<table>
<thead>
<tr>
<th>Pavement Temp (°C)</th>
<th>Representative Time Range</th>
<th>( (d_{\text{max}} - d_{200}) ) FWD (0.001 mm)</th>
<th>( (d_{\text{max}} - d_{200}) ) BB (0.001 mm)</th>
<th>( N_{\text{def}} ) (SS)</th>
<th>Traffic Loading (%)</th>
<th>Remaining Life (SS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 06:00</td>
<td>170.96</td>
<td>102.13</td>
<td>14,273,205</td>
<td>10.903</td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>10:00 - 16:00</td>
<td>219.59</td>
<td>122.66</td>
<td>6,280,166</td>
<td>44.997</td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>16:00 - 20:00</td>
<td>195.32</td>
<td>117.99</td>
<td>7,459,547</td>
<td>39.549</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 02:00</td>
<td>202.13</td>
<td>117.96</td>
<td>7,451,724</td>
<td>5.591</td>
<td></td>
</tr>
</tbody>
</table>

(c) Rutting Criterion

<table>
<thead>
<tr>
<th>Pavement Temp (°C)</th>
<th>Representative Time Range</th>
<th>Modules (MPa)</th>
<th>( N_{\text{retak}} ) (SS)</th>
<th>Traffic Loading (%)</th>
<th>Remaining Life (SS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 06:00</td>
<td>1,853.97</td>
<td>0.51</td>
<td>262.91</td>
<td>70.690</td>
</tr>
<tr>
<td>46</td>
<td>10:00 - 16:00</td>
<td>1,292.02</td>
<td>0.60</td>
<td>274.60</td>
<td>67.73</td>
</tr>
<tr>
<td>52</td>
<td>16:00 - 20:00</td>
<td>1,579.76</td>
<td>0.61</td>
<td>307.61</td>
<td>176.41</td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 02:00</td>
<td>1,506.50</td>
<td>0.65</td>
<td>217.65</td>
<td>170.48</td>
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</tbody>
</table>

(d) Fatigue Cracking Criterion

<table>
<thead>
<tr>
<th>Pavement Temp (°C)</th>
<th>Representative Time Range</th>
<th>Modules (MPa)</th>
<th>( N_{\text{skakr}} ) (SS)</th>
<th>Traffic Loading (%)</th>
<th>Remaining Life (SS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 06:00</td>
<td>1,853.97</td>
<td>0.51</td>
<td>262.91</td>
<td>70.690</td>
</tr>
<tr>
<td>46</td>
<td>10:00 - 16:00</td>
<td>1,292.02</td>
<td>0.60</td>
<td>274.60</td>
<td>67.73</td>
</tr>
<tr>
<td>52</td>
<td>16:00 - 20:00</td>
<td>1,579.76</td>
<td>0.61</td>
<td>307.61</td>
<td>176.41</td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 02:00</td>
<td>1,506.50</td>
<td>0.65</td>
<td>217.65</td>
<td>170.48</td>
</tr>
</tbody>
</table>

The values of \( N_{\text{dmax}}, N_{d200}, N_{\text{def}}, N_{\text{retak}} \) represent the structural capacity of an existing pavement based upon each design criterion. The smallest is normally reported as the remaining life of the pavement.
8. PAVEMENT STRUCTURAL CAPACITY

Substituting deflection data directly into Equations (3a) and (3b), and using back calculated modulus as input into program PastDean (Kosasih, 2004) for calculating the strains which are in turn entered into Equations (3c) dan (3d), the remaining life of the existing pavement can be effected.

Miners’ Law is then employed to calculate the combined effects of the four pavement conditions observed, each representing 6 hour interval in a day. Table 2 summarizes the calculation of the remaining life of the pavement at STA 0+000 by applying the four design criteria. The composition of traffic loadings within each time interval was obtained from past studies. As seen in the table, fatigue cracking criterion dictates the remaining life of the pavement of 0.2014 mill ESA.

The other two pavement sections at STA 0+500 and STA 1+000 are also governed by the fatigue cracking criterion with the remaining life of 0.2862 mill ESA and 0.2303 mill ESA respectively.

As discussed earlier, the pavement structure at STA 0+000 although showing the smallest maximum deflection (see Figure 2a) has the shortest remaining life.

<table>
<thead>
<tr>
<th>Pavement Temp (°C)</th>
<th>Representative Time Range</th>
<th>Modulus, E (MPa), for Layer Thickness, H (mm):</th>
<th>N&lt;sub&gt;max&lt;/sub&gt; (SS)</th>
<th>Traffic Loading (%)</th>
<th>N&lt;sub&gt;max&lt;/sub&gt; (SS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>02:00 - 06:00</td>
<td>E&lt;sub&gt;1&lt;/sub&gt; = 142, E&lt;sub&gt;2&lt;/sub&gt; = 111, E&lt;sub&gt;3&lt;/sub&gt; = 1, E&lt;sub&gt;4&lt;/sub&gt; = 0.94, H&lt;sub&gt;1&lt;/sub&gt; = 170.64, H&lt;sub&gt;2&lt;/sub&gt; = 200.91</td>
<td>6,198,250</td>
<td>10.865</td>
<td>10.865</td>
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<tr>
<td>46</td>
<td>10:00 - 16:00</td>
<td>E&lt;sub&gt;1&lt;/sub&gt; = 123, E&lt;sub&gt;2&lt;/sub&gt; = 126, E&lt;sub&gt;3&lt;/sub&gt; = 0.60, H&lt;sub&gt;1&lt;/sub&gt; = 170.64, H&lt;sub&gt;2&lt;/sub&gt; = 274.65</td>
<td>4,319,100</td>
<td>44.987</td>
<td>5,098,516</td>
</tr>
<tr>
<td>32</td>
<td>08:00 - 10:00</td>
<td>E&lt;sub&gt;1&lt;/sub&gt; = 157.76, E&lt;sub&gt;2&lt;/sub&gt; = 157.76, H&lt;sub&gt;1&lt;/sub&gt; = 207.81, H&lt;sub&gt;2&lt;/sub&gt; = 176.41</td>
<td>5,838,200</td>
<td>38.549</td>
<td>38.549</td>
</tr>
<tr>
<td>24</td>
<td>20:00 - 22:00</td>
<td>E&lt;sub&gt;1&lt;/sub&gt; = 150.69, E&lt;sub&gt;2&lt;/sub&gt; = 150.69, H&lt;sub&gt;1&lt;/sub&gt; = 217.65, H&lt;sub&gt;2&lt;/sub&gt; = 170.43</td>
<td>5,391,700</td>
<td>5.581</td>
<td>5.581</td>
</tr>
</tbody>
</table>

From visual observation, the pavement has also shown signs of distresses. If this pavement is to be overlaid, say to carry future 5 mill ESA, the overlay thickness required by the pavement at the three stations is 142 mm, 114 mm and 128 mm. Table 3 summarizes the calculation of overlay thickness requirement for the pavement at STA 0+000. Again, the pavement at STA 0.000 that has the weakest condition requires the thickest design overlay.

10. CONCLUSION

1. In a back calculation process, temperature correction factor and seasonal correction factor on deflection data are not required. These factors are taken into account in the resulting pavement layer modulus.

2. The asphalt layer modulus resulting from back calculation is stress-dependent. This allows variations in pavement conditions including traffic loadings to be analyzed by using Miners’ law.

3. Back calculation process for a pavement modeled as a four-layered system including a bonding layer may result in a more logical pavement layer modulus.

4. The resulting back calculated pavement layer modulus can be used directly to assess the structural capacity of a pavement by employing analytical design principles.
REFERENCES


