ENHANCING THE PREDICTION OF SUBGRADE STIFFNESS MODULUS AND CBR USING FWD FOR FLEXIBLE PAVEMENTS

GARY W. CHAI a, S.H. CHOWDHURY b and S. MANOHARAN c
a,b Griffith School of Engineering, Griffith University, Queensland, Australia
b Email: g.chai@griffith.edu.au
b Email: s.h.chowdhury@griffith.edu.au
c Queensland Department of Transport & Main Roads, Brisbane, Queensland, Australia,
c Email: sittam.p.manoharan@tmr.qld.gov.au

ABSTRACT

Deflection based model developed by Queensland Department of Transportation and Main Roads (TMR) is commonly used for predicting the subgrade CBR of asphalt pavements. The model utilizes Falling Weight Deflectometer (FWD) deflection $D_{900}$ recorded at 900 mm from the impact load. The principal aim of the study is to enhance the prediction of in-situ subgrade stiffness modulus and CBR using FWD for asphalt pavements with thin surfacing layers. The scope of the study included the comparison of subgrade CBR predictions from the deflection model and the predictions were verified using the in-situ CBR values derived from Dynamic Cone Penetrometer (DCP) from eleven pavement test sites. The approach for computing the degree of nonlinearity of subgrade was discussed. The study shows that the deflection model over predicts the subgrade CBR because the deflections recorded at sensor $D_{900}$ are consistently small due to the nonlinearity of the subgrade material. Subsequently, a modified TMR model was developed, by considering subgrade nonlinearity in the model. As a result, the CBR prediction was significantly enhanced.

Keywords: CBR, Falling Weight Deflectometer, Non-linearity of subgrade.

1. INTRODUCTION

Deflection based models developed by Jameson (1993), Roberts et al (2006) and Queensland Department of Transportation and Main Roads (TMR, 1993) are commonly used for predicting the subgrade CBR in Southeast Queensland (SEQ). The models utilise FWD deflection data recorded at sensor $D_{900}$ from the impact load. A study by Chai et al (2013) shows that the three deflection based models over predict the subgrade CBR because of the relatively small deflections are recorded at sensor $D_{900}$. As such, the three models which use $D_{900}$ deflection data were found to be not suitable for use in predicting the subgrade CBR for thin bituminous pavements when small deflection ($<0.100$ mm) is recorded at $D_{900}$. Utilising the $D_{450}$ deflection data, the study shows that the deflection data at $D_{450}$ yielded more reliable results and provided an enhanced prediction of subgrade CBR for thin bituminous pavements with asphalt layers less than 50 mm. It was recommended that further enhancement of the model be carried out with additional field data through the various stages of the development.

This paper is an extension of the previous research work conducted by Chai et al (2013). The principal aim of the study is to enhance predictions of subgrade CBR for flexible pavements using the $D_{900}$ deflection data. The scope of the study include:

- Modifying the TMR model by incorporating subgrade nonlinearity in the model; and
- Validating the predictions of the modified TMR model using the in-situ CBR values inferred from the Dynamic Cone Penetrometer (DCP).
2. SUBGRADE NONLINEARITY

The nonlinearity behaviour of the subgrade material is analysed by computing the Surface Modulus using Boussinesq’s equations (Ullidzt, 1987) as shown below. The maximum deflection ($D_0$) underneath the centre of the load was used to compute the subgrade Surface Modulus as shown in Equation 1 and the deflection recorded at sensors $D_{200}$, $D_{300}$, $D_{450}$, $D_{600}$, $D_{900}$ and $D_{1500}$ were used for calculating the Surface Modulus as in Equation 2.

\[
E_0(0) = \frac{2(1-\nu^2)\times R \times \sigma_0}{D_0}\]

\[
E_0(r) = \frac{(1-\nu^2)\times R^2 \times \sigma_0}{r \times D_r}
\]

where, $E_0(0)$ is the subgrade surface modulus at the center of the load (MPa). $E_0(r)$ is the subgrade stiffness modulus at distance $r$ (MPa). $\nu$ is the poison ratio (0.35) and $R$ is the radius of the plate. $\sigma_0$ is the constant pressure (kPa) and $r$ is the distance from the center of the loading plate (mm). $D_0$ is the deflection underneath the centre of the loading plate (mm) and $D_r$ is the deflection at distance $r$ (mm).

The exponential $n$ value is the material constant and is taken as a measurement of the nonlinearity of subgrade. If $n$ is computed to be zero the material is said to be linear elastic. As $n$ decreases toward a larger negative value between -0.1 and -1, the nonlinearity of the subgrade material becomes more and more pronounced. The equation to be used for computing $n$ is as follows (Ullidzt, 1998):

\[
n = \frac{\log \left( \frac{E_0(r_1)}{E_0(r_2)} \right)}{2 \log \left( \frac{r_2}{r_1} \right)}
\]

where, $n$ is material constant and $E_0(r_1)$ is the subgrade surface modulus at a distance $r_1$ (MPa). $E_0(r_2)$ is the subgrade stiffness modulus at distance $r_2$ (MPa). $r_1$ is the distance $r_1$ from the center of the loading plate (mm) and $r_2$ is the distance $r_2$ from the center of the loading plate (mm). $E_0(r)$ is the surface modulus at a distance $r$ larger than the equivalent thickness of the pavement. In the equivalent layer theory of Odemark (1949), overlaying (pavement construction) layers with different thicknesses and moduli are combined into one layer with an equivalent thickness.

Subgrade nonlinear behavior of thin surfaced flexible pavements can also be analyzed using Simplified Deflection Modeling (SDM) (Chai et al, 2016). In the studies carried out by Chai & Kelly (2008) and Chai et al. (2010), it was found that FWD deflection data obtained from the South East Queensland’s Long-Term Pavement Performance (SEQ-LTPP) sites can be modelled accurately using SDM. The exponential curve in SDM was found to have the desired characteristics that match the FWD deflection bowls. The parameters used in the model are explained as follows:

\[
Y_r = K_1 e^{(-r/K_2)}
\]

where $Y_r$ is the FWD deflection at the respective sensor location (micron) and $r$ is the respective sensor offset location (millimetres). $K_1$ is equal to deflection at $D_0$ in micron and $K_2$ is the structural
parameter at the respective sensor location. For deflection at sensor D$_{900}$, the equation becomes:

$$Y_{900} = K_1 e^{(-900/K_{2,900})}$$  \hspace{1cm} (5)$$

$$K_{2,900} = \frac{-900}{(\log e D_{900} - \log e D_0)}$$  \hspace{1cm} (6)$$

where $Y_{900}$ is the FWD deflection at the sensor location D$_{900}$ (micron), $K_{2,900}$ is the $K$ parameter for FWD deflection at D$_{900}$ and D$_0$ is the center deflection. The parameter $K_{2,900}$ is found to have a direct relationship with the material constant, $n$ value and the parameter can be used as a measurement of the degree of nonlinearity. The relationship between $K_{2,900}$ parameters and $n$ values has been developed and the equation can be expressed as follows:

$$K_{2,900} = 435 e^{(n/0.4546)} + 143$$  \hspace{1cm} (7)$$

where $K_{2,900}$ is the $K$ parameter for FWD deflection at sensor location D$_{900}$ (micron) and $n$ is the degree of nonlinearity. When the degree of nonlinearity, $n$ value is -0.50, $K_{2,900}$ for the particular deflection basin is computed to be 288. As the degree of nonlinearity increases to -1.00, $K_{2,900}$ decreases to 191. It can be observed that when $K_{2,900}$ is smaller than 300, the deflection basin is associated with high degree of subgrade nonlinearity. For moderate degree of nonlinearity, $K_{2,900}$ falls within the range of between 300 to 500. As $K_{2,900}$ increases and approaches 500, the pavement structure is observed to possess linear elastic behaviour with $n$ value is nearly equal to zero. Using the newly developed relationship, the degree of subgrade nonlinearity can be defined using the $K_{2,900}$ and $n$ value as shown in Table 1.

<table>
<thead>
<tr>
<th>Degree of subgrade nonlinearity</th>
<th>$K_{2,900}$</th>
<th>$n$-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>&gt;500</td>
<td>0.00</td>
</tr>
<tr>
<td>Moderate</td>
<td>300 to 500</td>
<td>0.00 to -0.50</td>
</tr>
<tr>
<td>High</td>
<td>&lt;300</td>
<td>&lt;-0.50</td>
</tr>
</tbody>
</table>

3. METHODOLOGY

DCP and FWD were conducted to assess the subgrade CBR of the eleven pavement test sections selected from the road network in Brisbane City. Thirty DCP test points were carried out at thirty FWD test locations along the test sections. From pavement coring, the thickness of the asphalt layers was found to be between 20 to 50mm. The granular base layers vary from 160 to 200 mm in thickness. For the pavements, FWD deflection basins were measured and reported at distances of 0, 200, 300, 450, 600, 900 and 1500 mm from the centre of the test load. These deflections are denoted as D$_0$, D$_{200}$, D$_{300}$, D$_{450}$, D$_{600}$, D$_{900}$ and D$_{1500}$ respectively. The deflections as far as possible from the centre of the applied load are recommended and preferably up to 1500 mm offset distance. The deflections at large offsets would allow a good presentation of a full extent of the deflection basin. In this study, deflection D$_{900}$ was used in estimating the subgrade CBR by using the three models specified in equations 1, 2 and 3. A new predictive model was developed utilising the corrected FWD D$_{900}$ deflection data at 900 mm from centre of loading plate.

The target FWD test load was 50 kN and with a standard 300 mm diameter loading plate, the corresponding surface stress was 700 kPa. The measured deflections were then ‘normalised’ to the appropriate surface stress, to correspond with operating tyre pressures. Deflections from FWD testing
were ‘normalised’ to the relevant target load by multiplying the measured deflections by the ratio of the target load to the actual load.

Dynamic Cone Penetrometer (DCP) is the direct field method used to estimate the subgrade CBR for cohesive soils in accordance with Australian Standard AS 1289.6.3.2 (AS, 1997). As the penetration cone is driven through the subgrade layer of pavement, for each drop of the standard weight and the penetration is measured in mm/blow. Austroads (2008) presented the correlation between CBR value and DCP test for fine-grained cohesive soils as shown:

$$\text{CBR}_{\text{subgrade}} = 10^{(2.154 - 1.555 \log(\text{DCP}))}$$

A deflection based model was developed by the Queensland Department of Main Roads (TMR, 1993). It was found that the subgrade response is reflected at D\text{900} and is relatively independent of the pavement structure of overlying pavement. For pavements without bound, thick asphalt or rigid layers, the D\text{900} deflection has been found to reflect a subgrade response that remains essentially unaffected by the structure of the overlying pavement and has been used to estimate the subgrade CBR at the time of testing (TMR, 1993). This relationship is shown in equation 9.

$$\text{CBR}_{\text{subgrade}} = 0.5996(D_{\text{900}})^{-1.4543}$$

Where CBR\text{subgrade} = California Bearing Ratio of subgrade (%) and D\text{900} = the FWD deflection observed at 900 mm from the load centre (mm). A comparison of subgrade CBR predictions obtained from the deflection model and the in-situ CBR values derived from DCP was then carried out. The degree of subgrade nonlinearity for each of the FWD deflection bowl was computed using equation 6.

4. DISCUSSION OF RESULTS

Pavement coring was carried out at the FWD test point locations in the eleven test sections with an average of 2 to 3 boreholes per test section. Soil profiles from the boreholes indicate the pavements consist of 30-50mm asphalt over 165-250mm granular base layer. The subgrade layers for eight test sites consist predominantly of Clay with traces of silt and sand. According to the AASHTO Soil Classification System (AASHTO, 1991), the soil is classified as A-2-7 and is described as clayey sand. The Liquid Limits (LL) in the sites ranging from 48 to 68, Plastic Limits (PL) range from 22 to 27 and Plastic Index (PI) from 26 to 41. The moisture content of the subgrade ranges from 10.8 to 18.8%. The subgrade soil at Test Site No. 11 consists of high plasticity clay with PL, PI and LL of 15, 60 and 75 respectively. The soil is classified as A-7-5. Test Sites No.5 and 9 consist of silty sand with the AASHTO Soil Classification as A-2-4. The in-situ subgrade CBR values of the test sites were determined using the Austroads CBR-DCP model (Austroads, 2008) as shown in Eq. 8. The CBR values were determined to be between 3 to 23 percent (see Table 4). The subgrade CBR values are consistent with the subgrade soil types with CBR vary from 8 to 23% for clayey sand soil and 3% for clayey soil.

The deflection data generated by the FWD device at the eleven Test Sections (TS) are presented in Table 2. The D\text{0} deflection varies from 465 (0.465 mm) to 2,515 microns (2.515 mm). For D\text{450}, the deflection is reported to be between 110 (0.110 mm) to 734 microns (0.734 mm). Relatively small deflections were recorded at the D\text{900} sensors. At this sensor location, the deflection varies from 11 (0.011 mm) to 118 microns (0.118 mm) and is nearly identical despite the increase in the deflection in D\text{0}. These are particularly obvious for Test Sites No.1 to 6. The same trends were also observed for deflections at sensors D\text{60} and D\text{150}.

One reason for these consistently small deflections is the dynamic affect of the FWD load which influences mainly the pavement materials near the impact load at the time of contact. The deflection basins show that the radius of influence zone for the thin granular pavements (with bituminous layer less than 50 mm) is about 450 mm from the impact load. This distance is between 1.5 to 2.0 times the total thicknesses of pavement layers. This is evidenced from the deflections at D\text{200}, D\text{300} and D\text{450}.
which show an increase of deflection as the $D_0$ deflection increases. The deflections at $D_{600}$, $D_{900}$ and $D_{1500}$ do not show a similar pattern of response. The main observation from these deflection characteristics is that the deflection basins recorded at sensors beyond $D_{450}$ exhibit non-linearity behaviour. The degree of subgrade nonlinearity was computed using equation 6 and it varies from 218 to 392 as shown in Table 3. This indicates that the pavements presented in Sections 1 to 11 showing moderate to highly nonlinear behaviour.

### Table 2: FWD deflections at the Test Sites

<table>
<thead>
<tr>
<th>Test Sections</th>
<th>FWD deflection (micron or mm x 10$^{-3}$)</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>$D_0$</td>
</tr>
<tr>
<td>1</td>
<td>642</td>
</tr>
<tr>
<td>2</td>
<td>681</td>
</tr>
<tr>
<td>3</td>
<td>1044</td>
</tr>
<tr>
<td>4</td>
<td>523</td>
</tr>
<tr>
<td>5</td>
<td>465</td>
</tr>
<tr>
<td>6</td>
<td>653</td>
</tr>
<tr>
<td>7</td>
<td>1876</td>
</tr>
<tr>
<td>8</td>
<td>476</td>
</tr>
<tr>
<td>9</td>
<td>704</td>
</tr>
<tr>
<td>10</td>
<td>743</td>
</tr>
<tr>
<td>11</td>
<td>2515</td>
</tr>
</tbody>
</table>

### Table 3: Comparison of the subgrade CBR with different deflection based models

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Subgrade nonlinearity ($K_{2,900}$)</th>
<th>Subgrade CBR (%) derived from the deflection based models</th>
<th>AASHTO Soil Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>TMR (1993)</td>
<td>Modified TMR Model (Eq. 13)</td>
</tr>
<tr>
<td>1</td>
<td>263</td>
<td>165</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>218</td>
<td>423</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>227</td>
<td>177</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>311</td>
<td>103</td>
<td>21</td>
</tr>
<tr>
<td>5</td>
<td>355</td>
<td>73</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>316</td>
<td>69</td>
<td>15</td>
</tr>
<tr>
<td>7</td>
<td>289</td>
<td>22</td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>392</td>
<td>50</td>
<td>24</td>
</tr>
<tr>
<td>9</td>
<td>324</td>
<td>56</td>
<td>14</td>
</tr>
<tr>
<td>10</td>
<td>343</td>
<td>42</td>
<td>13</td>
</tr>
<tr>
<td>11</td>
<td>294</td>
<td>13</td>
<td>3</td>
</tr>
</tbody>
</table>

In view of the inherent characteristics of the deflection basins shown by the thin bituminous pavements, $D_{900}$ deflection from FWD would not be a reliable data for use in predicting the subgrade CBR. By incorporating subgrade nonlinearity in the original TMR deflection model, a modified TMR model has been developed. This can be achieved by setting $k_{2,900} = 500$ in equation (11) to correct the FWD deflection for subgrade nonlinearity. The $D_{900}$ (corrected) becomes $D_0 e^{(-900/k_{2,900})}$ and it can be further expressed as $D_{900}$ (corrected) = 0.1652*D$_0$. Substituting $D_{900}$ (corrected) = 0.1652*D$_0$ in Equation (10), the expression of the modified TMR model is presented as Equation (13) in terms of $D_0$:

$$CBR_{subgrade} = 0.5996 \left( D_{900} \text{ (corrected)} \right)^{1.4543}$$  \hspace{1cm} (10)

$$D_{900} \text{ (corrected)} = D_0 e^{(-900/k_{2,900})}$$  \hspace{1cm} (11)

$$D_{900} \text{ (corrected)} = 0.1652 \times D_0$$  \hspace{1cm} (12)
CBR_{subgrade} = 0.5996 (0.1652D_0)^{-1.4543} \quad (13)

E_{subgrade} = 5.996 (0.1652D_0)^{-1.4543} \quad (14)

Where \(D_0\) = the FWD deflection observed at the load centre (mm), \(CBR_{subgrade}\) = California Bearing Ratio of subgrade (%), \(E_{subgrade}\) = stiffness modulus of subgrade (MPa), \(D_{900}\) = the FWD deflection observed at 900 mm from the load centre (mm) and corrected \(D_{900}\) = corrected deflection \(D_{900}\) taking subgrade nonlinearity into consideration.

Figure 1 to 2 show the relationships between the subgrade CBR derived from the corrected \(D_{900}\) deflection versus the CBR obtained from DCP test. The graph in Figure 1 shows that the current TMR model (TMR, 1993) yielded an \(R^2\) value of 0.40 and relatively high RMSE value of 143.4 percent. In comparison, the modified TMR model (Equation 13) has an \(R^2\) value of 0.93 and a low RMSE value of 2.69 percent (see Figure 2).

The study shows that TMR models (1993) over predicted the CBR values by a sizable margin of errors because the deflection at \(D_{900}\) are consistently small which ranges from 11 to 54 micron (0.011 to 0.054 mm) in most of the test sites. When \(D_{900}\) recorded a reading of 11 micron, the original TMR predicted a subgrade CBR value of 423%. One reason for these consistently small deflections is the dynamic effect of the FWD load which influences mainly the materials of the thin asphalt pavement near the impact load (at distance equal or less than 450mm) at the time of contact. As such, the model uses uncorrected \(D_{900}\) deflection data, was found to be not suitable for predicting the subgrade CBR for thin bituminous pavements with moderate to high subgrade nonlinearity. The study shows that by correcting the deflection data at \(D_{900}\) for subgrade nonlinearity yielded more reliable results and provided an enhanced prediction of subgrade CBR for this pavement type.

![Graph showing comparison between subgrade CBR derived from TMR model and DCP results.](image)

Figure 1: Comparison of QDMR model’s prediction with DCP results
Figure 2: Comparison of modified TMR model’s prediction with DCP results

5. CONCLUSION

The research objectives outlined in the paper have been achieved. The characteristics of the FWD deflection basins for thin granular pavements with bituminous layer have been examined and the inherent properties of the deflection basins have been discussed. By incorporating the subgrade nonlinearity in the TMR (1993) model, the CBR prediction was significantly enhanced. The findings from the study are summarised as follows:

- It is observed that the subgrade nonlinearity is computed to be 250 to 314 and the degree of nonlinearity is classified as moderate to high.
- The study shows that the current TMR model (TMR, 1993) over predict the subgrade CBR because of the relatively small deflections are recorded at sensor D900. As such, the model which uses uncorrected D900 deflection data was found to be not suitable for use in predicting the subgrade CBR for thin bituminous pavements when small deflection (<0.100 mm) is recorded at D900.
- Utilising the corrected D900 deflection data, the study shows that the corrected deflection taking subgrade nonlinearity into consideration yielded more reliable results and provided an enhanced prediction of subgrade CBR.
- The modified TMR model has been successfully developed for application in asphalt pavements showing moderate to highly nonlinear subgrade behaviour.
ACKNOWLEDGEMENT

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