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

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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 utilises 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.

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

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

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). ν is the poison ratio (0.35) and R is the radius of the plate. σ_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):

$$n = \frac{\log\left(\frac{E_0(r_1)}{E_0(r_2)}\right)}{2 \times \log\left(\frac{r_2}{r_1}\right)} \tag{3}$$

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:

$$Y_r = K_1 e^{(-r/K_2)} \tag{4}$$

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:

$$Y_{900} = K_1 e^{\left(-900/K_{2,900}\right)} \tag{5}$$

$$K_{2,900} = \frac{-900}{(\log eD_{900} - \log eD_0)} \tag{6}$$

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:

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

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 1: Definition of Degree of Nonlinearity of Subgrade (Chai et al, 2016)

Degree of subgrade nonlinearity	$K_{2,900}$	*n*-value
Linear	>500	0.00
Moderate	300 to 500	0.00 to -0.50
High	<300	<-0.50

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:

$$CBR_{subgrade} = 10^{(2.154 - 1.555 \log10(DCP))} \quad (8)$$

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_{900} and is relatively independent of the pavement structure of overlying pavement. For pavements without bound, thick asphalt or rigid layers, the D_{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.

$$CBR_{subgrade} = 0.5996(D_{900})^{-1.4543} \quad (9)$$

Where $CBR_{subgrade}$ = California Bearing Ratio of subgrade (%) and D_{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_0 deflection varies from 465 (0.465 mm) to 2,515 microns (2.515 mm). For D_{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_{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_0. These are particularly obvious for Test Sites No.1 to 6. The same trends were also observed for deflections at sensors D_{600} and D_{1500}.

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 $D_{200,}$ D_{300} and D_{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

Test Sections	FWD deflection (micron or mm x 10^{-3})						
	D_0	D_{200}	D_{300}	D_{450}	D_{600}	D_{900}	D_{1500}
1	642	472	291	155	79	21	8
2	681	479	310	110	51	11	10
3	1044	719	515	264	70	20	17
4	523	383	259	145	83	29	13
5	465	320	222	132	81	37	21
6	653	464	322	179	77	38	23
7	1876	1278	786	314	116	84	69
8	476	350	274	189	126	48	18
9	704	467	317	170	93	44	27
10	743	573	403	233	136	54	25
11	2515	1902	1363	734	348	118	88

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

Test Section	Subgrade nonlinearity ($K_{2,900}$)	Subgrade CBR (%) derived from the deflection based models			AASHTO Soil Classification
		TMR (1993)	Modified TMR Model (Eq. 13)	In-situ CBR from DCP	
1	263	165	16	16	A-2-7
2	218	423	15	23	A-2-7
3	227	177	8	12	A-2-7
4	311	103	21	20	A-2-7
5	355	73	25	20	A-2-4
6	316	69	15	15	A-2-7
7	289	22	4	8	A-2-7
8	392	50	24	13	A-2-7
9	324	56	14	16	A-2-4
10	343	42	13	13	A-2-7
11	294	13	3	3	A-7-5

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 $D_0\, e^{(-900/500)}$ 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:

$$CBR_{subgrade} = 0.5996\,(D_{900(corrected)})^{-1.4543} \tag{10}$$

$$D_{900\ (corrected)} = D_0\, e^{\ (-900/k2,900)} \tag{11}$$

$$D_{900\ (corrected)} = 0.1652*D_0 \tag{12}$$

$$CBR_{subgrade} = 0.5996\ (0.1652 * D_0)^{-1.4543} \qquad (13)$$

$$E_{subgrade} = 5.996\ (0.1652 * D_0)^{-1.4543} \qquad (14)$$

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.

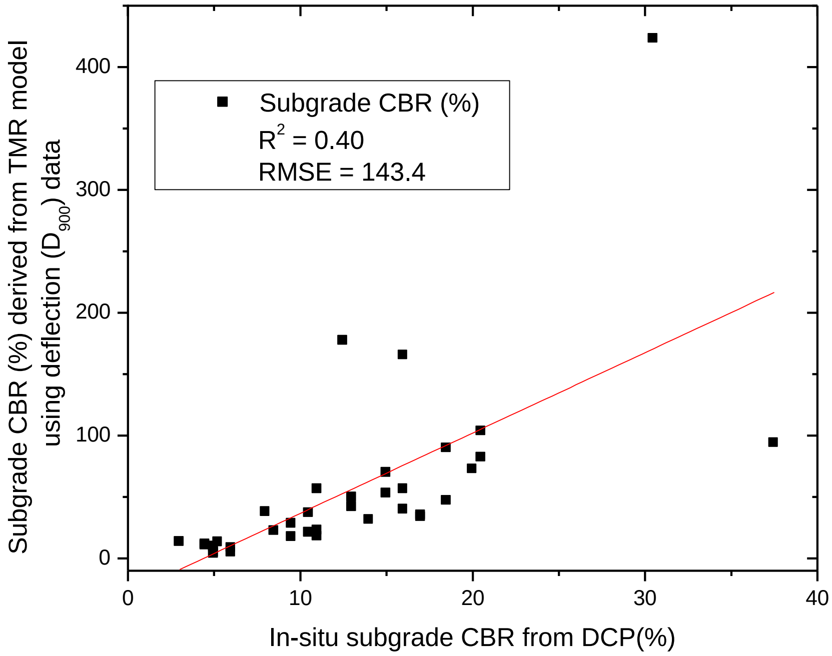

Figure 1: Comparison of QDMR model's prediction with DCP results

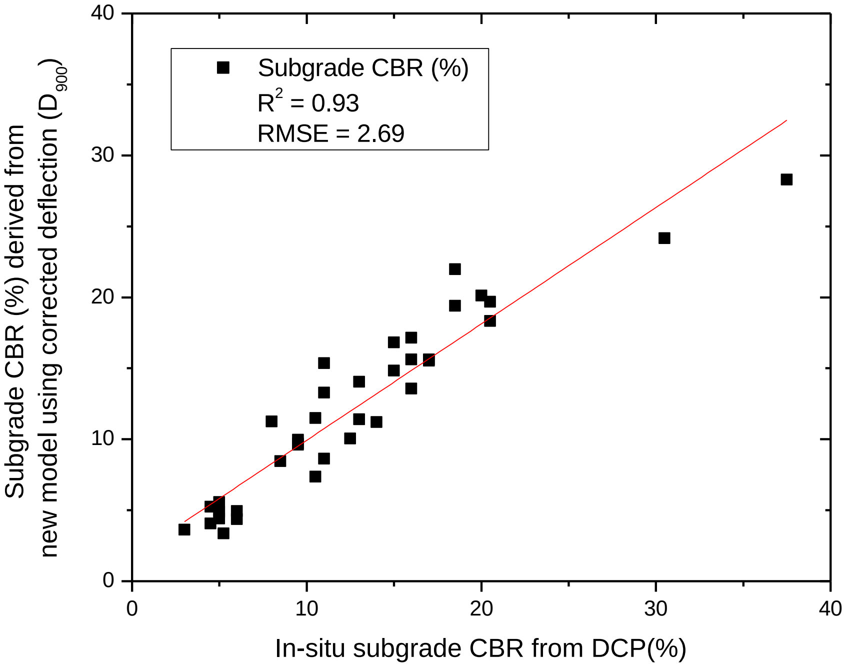

Figure 2: Comparison of modified TMR model's prediction with DCP results

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 $D_{900;}$ As such, the model which uses uncorrected D_{900} 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 D_{900}.
- Utilising the corrected D_{900} 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.

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REFERENCES

AASHTO. (1991) AASHTO M 145: Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, *American Association of State Highway and Transportation Officials*, 1991.

AASHTO. (1991) AASHTO M 145: Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, American Association of State Highway and Transportation Officials.

AS. (1997) Australia Standard AS 1289 6.3.2–1997, Methods for Testing Soils for Engineering Purposes: Soil Strength and Consolidation Tests: Determination of the Penetration Resistance of a Soil: 9 kg Dynamic Cone Penetrometer Test.

Austroads. (2008) Pavement Design – *A Guide to the Structural Design of Road Pavements*, Austroads Incorporated, Sydney, New South Wales, Australia.

Chai, G., and Kelly, G. (2008) Structural Characterisation of LTPP Pavements using FWD. *6th International Conference on Road & Airfield Pavement Technology*, Sapporo, Japan, 2008.

Chai, G., Lewer, G., and Cancian G. (2010). A Study of the FWD Deflection Characteristics of Composite and Sandwich Pavements. *11th International Conference on Asphalt Pavements*, Nagoya, Japan.

Chai, G., S. Argadiba, G. Stephenson, I. Condric, E. Oh, and S. Manoharan. (2013) Prediction of Subgrade CBR using FWD for Thin Bituminous Pavements, *International Journal of Pavement Research and Technology*, Vol.6, No. 4 and 5, 2013, pp. 280-286.

Chai, G., Kelly, G., Huang, A., Chowdhury, S.H., Manoharan, S., & Golding, A. (2016) New Approaches for Modeling Non-linearity of Subgrade in asphalt pavement, *International Journal of Pavement Engineering*, Taylor & Francis Group, United Kingdom.

Jameson, G.W. (1993) Development of procedures to predict structural number and subgrade strength from falling weight deflectometer deflections, *ARRB Transport Research*, Vermont South, Victoria.

Roberts, J., Michel, N., & Paine, D. (2006) STEP: A New Estimation of Flexible Pavement Configuration & Remaining Structural Life, *Road and Transport Research*, Vol. 15 No. 4, 2006.

Odemark, N. (1949) Investigations to the Elastic Properties of the Soil and the Design of Pavements to the Theory of Elasticity, Stockholm.

TMR. (1993) Pavement Rehabilitation Manual, *Queensland Department of Main Roads*, Brisbane, Queensland, 1993.

TMR. (2012) Pavement Rehabilitation Manual: Chapter 2 - Pavement Evaluation, *Queensland Department of Main Roads, Pavements Materials & Geotechnical Branch*, Brisbane, 2012

Ullidtz, P. (1987) Pavement Analysis, *Elsevier Science Publishing Company Inc*, 1987.

Ullidtz, P. (1998) Modelling Flexible Pavement Response and Performance. *Narayana Press*, Gylling, Denmark.