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Condition Evaluation of the Pavement Foundations Using Multi-load Level FWD Deflections

다단계 하중 FWD를 사용한 도로기초 상태평가 연구

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요 지

본 연구에서는 다단계 하중 Falling Weight Deflectometer를 사용하여 도로기초의 상태를 평가하는 방법을 제시하였다. 응력의존 재료모델을 포함한 동적 유한요소법을 활용하여 가상의 처짐과 응력/변형률 데이터베이스를 구축하였다. 이러한 가상의 데이터베이스를 바탕으로 표면처짐 및 보조기층 또는 노상의 중요한 위치에서 발생하는 응력/변형률과의 관계를 제시하였다. 미국의 LTPP와 노스캐롤라이나 주도로국에서 실시한 FWD 처짐값, 동적관입시험 결과, 그리고 반복하중 탄성계수시험을 활용하여 평가방법을 개발하였다. 특히 본 연구는 FWD 하중크기가 상태평가 방법에 미치는 영향에 대하여 연구의 초점을 맞추었다. 연구결과, 구조적으로 수정된 보조기층 손상지수와 보조기층 곡률지수가 각각 보조기층과 노상토의 강성도 특성을 예측할 수 있는 좋은 인자들로 판단되었다. 66.7kN 또는 그 보다 작은 하중은 예측의 정밀도를 높이는데 부족하였다. 도로기초의 비선형거동에 대한 연구결과, 다단계 하중으로부터 발생하는 처짐비는 도로기초 재료의 종류와 상태를 판단할 수 있었다.

Abstract

A condition evaluation procedure for the pavement foundations using multi-load level Falling Weight Deflectometer (FWD) deflections is presented in this paper. A dynamic finite element program, incorporating a stress-dependent material model, was used to generate the synthetic deflection database. Based on this synthetic database, the relationships between surface deflections and critical responses, such as stresses and strains in base and subgrade layers, have been established. FWD deflection data, Dynamic Cone Penetrometer (DCP) data, and repeated load resilient modulus testing results used in developing this procedure were collected from the Long Term Pavement Performance (LTPP) and North Carolina Department of Transportation (NCDOT) database. Research effort focused on investigation of the effect of the FWD load level on the condition evaluation procedures. The results indicate that the proposed procedure can estimate the pavement foundation conditions. It is also found that structurally adjusted Base Damage Index (BDI) and Base Curvature Index (BCI) are good indicators for the prediction of stiffness characteristics of aggregate base and subgrade respectively. A FWD test with a load of 66.7 kN or less does not improve the accuracy of this procedure. Results from the study for the nonlinear behavior of a pavement foundations indicate that the deflection ratio obtained from multi-load level deflections can predict the type and quality of the pavement foundation materials.

Keywords : Base curvature index, Base damage index, Dynamic cone penetrometer, Falling weight deflectometer, Pavement foundations

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1. Introduction

Falling Weight Deflectometer (FWD) deflections and Deflection Basin Parameters (DBPs) have been successfully used to estimate the pavement structural capacity and the current condition of existing pavements. In addition to the deflection basin parameters, the pavement responses at critical locations in each individual layer have proven to be good condition indicators for various distresses (Kim et al. (2000), Xu et al. (2002), and Garg and Thompson (1998)). These critical pavement responses can be predicted from the deflection basin parameters and layer thicknesses based on the statistical regression approach using the synthetic database developed by the dynamic finite element program.

In this paper, a new pavement condition assessment procedure for foundation layers in flexible pavements is presented using multi-load level FWD deflections. Kim et al. (2000) and Xu et al. (2002) developed the pavement condition assessment procedure using 40 kN load level FWD deflection data. Since this procedure was developed from a single load level test, it was deemed important to check the effect of load level on the condition assessment procedure. FWD deflection data, Dynamic Cone Penetrometer (DCP) data, and resilient modulus testing results for foundation materials were used in the validation of proposed procedure. This paper also presents the relationship between the nonlinear behavior of a pavement structure and the quality and type of pavement materials.

2. Synthetic Pavement Response Database

Synthetic pavement responses were computed using the ABAQUS finite element commercial software package for the dynamic analysis in full depth and aggregate base pavements. The 40kN, 53.3kN, and 66.7kN of load level were used for synthetic database generation. After surveying the database in Data Pave 2.0, the range of thickness of each pavement type was determined to cover as many existing pavements as possible.

To simulate the nonlinear behavior in base and subgrade materials, the universal soil model was implemented in these two finite element programs. The model parameters for granular materials were selected using information from the research of Garg and Thompson (1998), and the model parameters for subgrade soils were adopted from Santha (1994).

In this study, the synthetic database generated by the ABAQUS program was used in developing the pavement response models. Table 1 illustrates the range of layer thicknesses and moduli of pavement materials used in creating the nonlinear elastic synthetic database. A total of 2,000 cases for full-depth pavements and 8,000 cases for aggregate base pavements were generated using the random selection approach. The nonlinear elastic synthetic database includes the surface deflections at various offset distances from the center of the loading plate, and stresses and strains at specific locations in each individual layer.

3. Determination of Pavement Condition Indicators

The synthetic database mentioned previously was analyzed to identify deflection basin parameters that have a significant influence on the prediction of critical pavement responses in flexible pavements. All the deflection basin parameters used in this study are shown

Table 1. Nonlinear elastic synthetic database structures

Pavement Type	Pavement Layer	Thickness (mm)	Modulus (MPa)
Aggregate Base Pavement	Asphalt Concrete	51-610	690-11032
	Aggregate Base	52-610	*
	Subgrade	762-6096	**
Full Depth Pavement	Asphalt Concrete	51-711	690-16548
	Subgrade	762-6096	**

* after Garg and Thompson, 1998
** after Santha, 1994

in Figure 1. Among these, deflection basin parameters under a 40 kN load level were used in a parametric sensitivity analysis.

The correlations between DBPs and critical pavement responses were analyzed and Root Mean Square Error (RMSE) values were calculated for each DBP. Tables 2 and 3 show the results of the parametric sensitivity analysis for the full-depth pavement and the aggregate base pavement, respectively. The DBPs with the highest RMSEs marked in these tables were considered the best parameters for critical pavement response prediction.

According to Kim et al. (2000), the granular base materials influence only a small portion of pavement surface deflections. However, the condition of the base layer has a significant effect on the long-term performance of flexible pavements. For aggregate base pavements, it was found from the sensitivity analysis that the BDI is the most critical deflection parameter for the prediction of ε_{abc}. In addition, the difference in ε_{abc} values under 40 and 67 kN loads was also predicted using difference

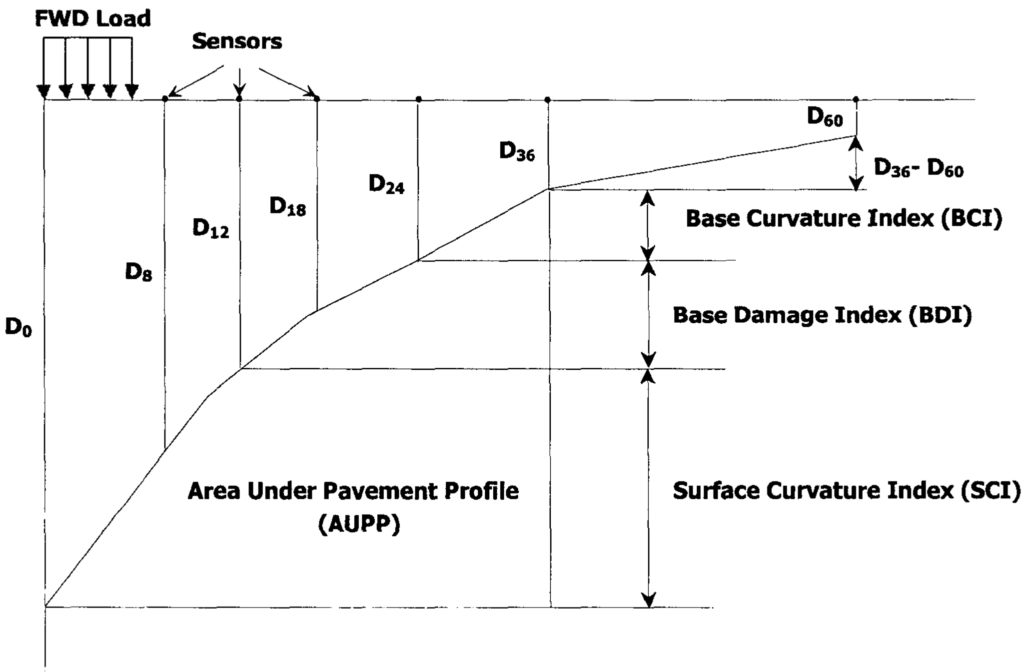


Fig. 1. Deflection basin parameters from a Falling Weight Deflectometer

Table 2. Parametric analysis results for full-depth pavements

Distress Type	Critical Response	DBP's	R Square
Fatigue Cracking	Tensile Strain at Bottom of AC layer	**BDI √**	**0.9858**
		AUPP √	**0.9530**
		BCI	0.9366
		SCI	0.8561
Rutting	Average Compressive Strain in AC layer	**SCI √**	**0.9110**
		AUPP	0.7476
		BDI	0.5206
		BCI	0.4182
	Compressive Strain on Top of Subgrade	**BDI √**	**0.9787**
		AUPP	0.9384
		BCI	0.9158
		SCI	0.8442
		$D_{36}-D_{60}$	0.5574

Table 3. Parametric analysis results for aggregate base pavements

Distress Type	Critical Response	DBP's	R Square
Fatigue Cracking	Tensile Strain at Bottom of AC layer	**BDI √**	**0.9808**
		AUPP √	**0.9319**
		BCI	0.9302
		SCI	0.8458
Rutting	Average Compressive Strain in AC layer	**SCI √**	**0.9110**
		AUPP	0.7476
		BDI	0.5206
		BCI	0.4182
	Compressive Strain on Top of Base Layer	**BDI √**	**0.9675**
		BCI	0.908
		AUPP	0.8824
		SCI	0.7830
		$D_{36}-D_{60}$	0.5155
	Compressive Strain on Top of Subgrade	**BCI √**	**0.7461**
		BDI	0.7157
		$D_{36}-D_{60}$	0.6240
		SCI	0.5320
		AUPP	0.4977

in BDI and DBDI values. The pavement response models for ε_{abc} and $d\varepsilon_{abc}$ are expressed as:

$$\log(\varepsilon_{abc}) = 0.938\log(BDI) - 0.0791\log(H_{ac}) + 0.045\log(H_{base}) + 3.826$$
$$R^2 = 0.970 \qquad SEE = 0.066 \tag{1}$$

$$\log(d\varepsilon_{abc}) = 0.918\log(DBDI) - 0.0071\log(H_{ac}) + 0.071\log(H_{base}) + 3.386$$
$$R^2 = 0.961 \qquad SEE = 0.067 \tag{2}$$

where H_{base} is the thickness of the base layer in mm.

From the AASHTO Guide (1993), a simple formula is presented for backcalculating the subgrade modulus from a single deflection measured from an outer-most sensor and the load magnitude. However, this approach may not be suitable for an accurate prediction of the stiffness of the subgrade because the load spreadability is a function of layer stiffness, distress condition, and thickness (Lee, 1997). For example, since there are no intermediate support layers in full depth pavements, the BDI and DBDI were found to be critical deflection basin parameters in predicting the compressive strain on the top of the subgrade, ε_{sg}, and the difference of ε_{sg} due to load level, $d\varepsilon_{sg}$, respectively. For full depth pavements, the ε_{sg} and $d\varepsilon_{sg}$ may be predicted using the following equations:

$$\log(\varepsilon_{sg}) = 0.999\log(BDI) + 0.0631\log(H_{ac}) + 3.583$$
$$R^2 = 0.979 \qquad SEE = 0.061 \tag{3}$$

$$\log(d\varepsilon_{sg}) = 1.000\log(DBDI) + 0.1031\log(H_{ac}) + 3.668$$
$$R^2 = 0.978 \qquad SEE = 0.062 \tag{4}$$

According to the parametric sensitivity study, instead of deflection at the outer-most sensor location, the Base Curvature Index (BCI) was found to be a good indicator of the condition of the subgrade for aggregate base pavements. The BCI is defined as the difference in deflections at 305 and 914 mm of the radial distance from the center of the load plate. The BCI value and the thicknesses of the AC and base layers were input to the pavement response model to predict the ε_{sg} value for aggregate base pavements. The difference of BCI values, the DBCI, obtained from deflections under different load levels also was investigated to predict the difference of ε_{sg} due to load level ($d\varepsilon_{sg}$). Similar to the full depth pavement, the ε_{sg} and $d\varepsilon_{sg}$ for aggregate base pavements can be calculated using the following equations:

$$\log(\varepsilon_{sg}) = 1.017\log(BCI) - 0.0421\log(H_{ac}) - 0.494\log(H_{base}) + 5.072$$
$$R^2 = 0.903 \qquad SEE = 0.125 \tag{5}$$

$$\log(d\varepsilon_{sg}) = 1.023\log(DBCI) - 0.0451\log(H_{ac}) - 0.445\log(H_{base}) + 4.928$$
$$R^2 = 0.909 \qquad SEE = 0.115 \tag{6}$$

4. Structural Correction Procedure for Pavement Layer Condition Assessment

The DBPs and critical pavement responses are strongly related to pavement layer condition, their values, however, are also dependent on structural and material properties in a flexible pavement. Kim et al. (2000) and Xu et al. (2002) proposed the structural correction procedure that normalizes these condition indicator values to a standard pavement structure. The standard structure used in this study is as follows: H_{ac} = 152.4 mm, E_{ac} = 3447 MPa, H_{base} = 25.4 mm., and H_{sg} = infinity. The condition indicators are described using structural and material properties of a flexible pavement. For example, the following regression equations for the base layer can be obtained using the synthetic database:

$$\log(BDI) = -1.549\log(H_{ac}) - 0.095\log(H_{base}) - 0.572\log(E_{ac}) - 0.013\log(E_{ri}) + 4.702$$
$$R^2 = 0.947 \qquad SEE = 0.090 \tag{7}$$

$$\log(DBDI) = -1.476\log(H_{ac}) - 0.112\log(H_{base}) - 0.559\log(E_{ac}) - 0.018\log(E_{ri}) + 4.352$$
$$R^2 = 0.935 \qquad SEE = 0.097 \tag{8}$$

$$\log(\varepsilon_{abc}) = -1.583\log(H_{ac}) - 0.001\log(H_{base}) - 0.591\log(E_{ac}) - 0.146\log(E_{ri}) + 8.064$$
$$R^2 = 0.940 \qquad SEE = 0.100 \tag{9}$$

$$\log(d\varepsilon_{abc}) = -1.362\log(H_{ac}) - 0.010\log(H_{base}) - 0.536\log(E_{ac}) - 0.145\log(E_{ri}) + 7.074$$
$$R^2 = 0.900 \qquad SEE = 0.124 \tag{10}$$

where E_{ri} is the subgrade modulus at 41kPa of deviatoric stress in MPa.

The adjusted BDI value corresponding to a standard pavement structure can be obtained by dividing the estimated BDI value obtained from an actual pavement by a structural correction factor.

$$Adjusted\ BDI = \frac{Estimated\ BDI}{\beta_1} \tag{11}$$

The structural correction factor, β_1 can be defined as follows.

$$\beta_1 = \frac{BDI_m}{BDI_r} \tag{12}$$

where BDI_r is the BDI value at a standard pavement structure, and BDI_m is the BDI value at an actual pavement structure.

5. Validation of Condition Assessment Procedure Using Dynamic Cone Penetrometer Data

Multi-load level FWD deflections and DCP testing results were collected from several test sections in the State of North Carolina, U.S. The load level used in FWD testing ranges from 26.7 to 53.3 kN. Results of DCP testing contain the number of weight drops and the penetration depth in the foundation layers. The California Bearing Ratio (CBR) value for each individual layer was estimated from the penetration depth per drop (PD) based on the empirical correlation developed by the NCDOT, as follows.

$$\log(CBR) = 2.6 - 1.07\log(PD) \tag{13}$$

It is noted that the DCP testing was only performed on the full depth pavements. To determine the thickness of the AC layer, coring data was also used.

Surface deflections and subgrade CBR values obtained from these pavement sections were incorporated to validate the procedure for condition assessment of the subgrade in full depth pavements. To evaluate the validity of this procedure, the CBR values in subgrade layer were compared with the predicted condition indicators. The relationships between adjusted BDI and DBDI values, and the subgrade CBR values in full depth pavements, are shown in Figure 2.

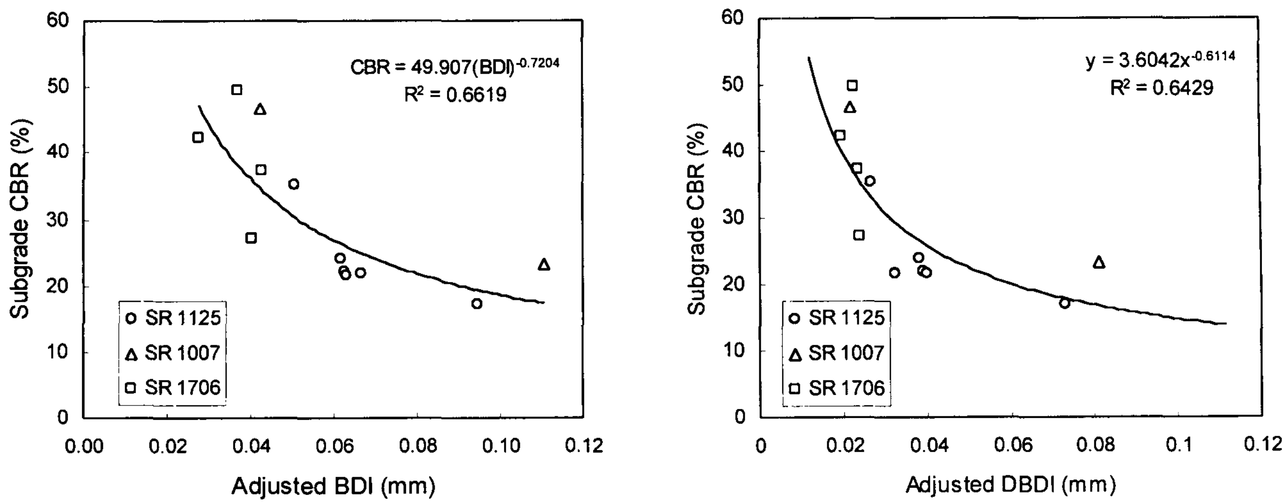


Fig. 2. Adjusted BDI and DBDI as a subgrade condition indicator for full depth pavements

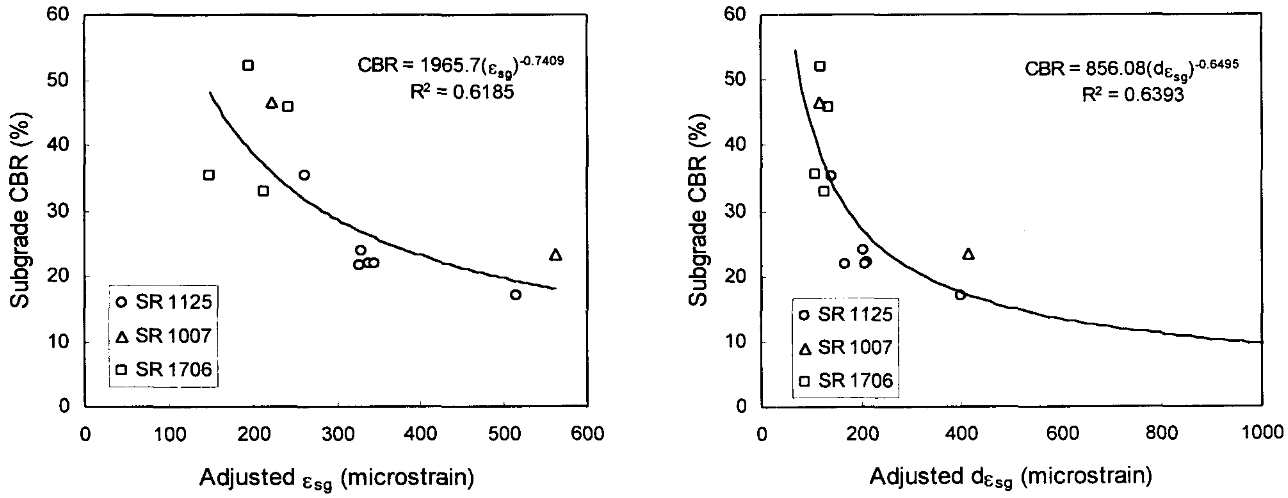


Fig. 3. Adjusted ε_{sg} and $d\varepsilon_{sg}$ as a subgrade condition indicator for full depth pavements

The first point to be made from these figures is the decreasing trend of the subgrade CBR values as the adjusted BDI and DBDI values increase. This finding is significant because the subgrade strength can be determined based on unique BDI/DBDI - subgrade CBR relationships. Another observation may be made by comparing the degree of correlation between the adjusted BDI and DBDI, and the subgrade CBR value. The degree of correlation for the BDI is slightly better than that for the DBDI. This finding indicates that the deflections under a 53.3 kN load level are not large enough to cause the significant nonlinearity in the behavior of subgrade soils and to assess the subgrade condition. Therefore, it is desirable to use higher load level deflection data for a more accurate condition assessment of the subgrade.

The predicted ε_{sg} and $d\varepsilon_{sg}$ values are plotted in Figure 3 against the subgrade CBR values. Similar trends to those indicated above were observed for deflection basin parameters. Although the $d\varepsilon_{sg}$ slightly improves the degree of correlation, the use of deflections under a 53.3 kN load level is still not satisfactory. This validation concludes that a higher FWD load (greater than 53.3 kN) is necessary to improve the accuracy in estimating the subgrade condition.

6. Validation of Condition Assessment Procedure Using Resilient Modulus Testing Data

The LTPP data in DataPave 2.0 (1999) were used to

Table 4. Characteristics of pavement test sections in LTPP data

State	SHRP ID	Thickness (mm)			Material Type		
		AC	Base	Subbase	Base	Subbase	Subgrade
NC (37)[1]	1028	266.7	139.7	–	Silty Sand	–	SM
TX (48)[1]	1077	129.5	264.2	–	Cr. Stone	–	ML
TX (48)[1]	1068	276.9	152.4	203.2	Cr. Stone	Lime-Tr Soil	CL
TX (48)[1]	1060	190.5	312.4	152.4	Cr. Stone	Lime-Tr Soil	SM
AL (1)[1]	0102	101.6	304.8	–	Cr. Stone	–	CL
CT (9)[2]	1803	182.9	304.8	–	Gravel	–	ML
MA (25)[2]	1002	198.1	101.6	213.4	Cr. Gravel	Soil Agg.	SP
MN (27)[2]	6251	188.0	259.1	–	Gravel	–	SP
NE (31)[2]	0114	177.8	304.8	–	Agg.		CL
NH (33)[2]	1001	213.4	490.2	365.8	Gravel	Soil Agg.	SP
OK (40)[2]	4165	68.6	137.2	–	HMAC	–	SM

[1] Wet no-freeze region
[2] Wet freeze region

Table 5. Confining pressures and deviator stresses used in the resilient modulus testing

Layer	Confining Pressure (kPa)	Deviator Stress (kPa)
Base	20.7	20.7, 41.4, 62.0
	34.5	34.5, 68.9, 103.4
	68.9	68.9, 137.9, 206.8
	103.4	68.9, 137.9, 206.8
	137.9	103.4, 137.9, 275.8
Subgrade	13.8	13.8, 27.6, 41.4, 55.2, 68.9
	27.6	13.8, 27.6, 41.4, 55.2, 68.9
	41.4	13.8, 27.6, 41.4, 55.2, 68.9

validate the proposed procedure in assessing pavement conditions in flexible pavements. All the test sections are located in wet no-freeze and wet freeze regions. Characteristics of the selected test sections are summarized in Table 4. The LTPP data for the validation procedure include the results of resilient modulus testing of base and subgrade materials and multi-load level FWD deflections. The average resilient moduli values of base and subgrade materials were measured for each sample cored from the LTPP test sections at selected confining pressures and deviator stresses. Table 5 shows the list of confining pressures and deviator stresses used in the resilient modulus testing. FWD deflections measured during the fall season of the year were used in this study.

The measured resilient moduli and corresponding bulk and deviator stress values were input to the universal soil model (Equation 14) to determine the coefficients k_1, k_2, and k_3 (Witzak and Uzan, 1988). In order to minimize the laboratory testing induced errors, the results of resilient modulus test were selected based on the criteria mentioned by Santha (1994). According to the criteria, this analysis was performed on the pavement sections with subgrade soils having decreasing resilient modulus with increasing deviator stress at lower deviator stresses and having increasing resilient modulus with increasing confining pressure. The predicted coefficients using the regression analysis are shown in Table 6 for the base and subgrade materials.

$$M_r = k_1(\theta)^{k2}(\sigma_d)^{k3} \tag{14}$$

where

θ = the sum of the principal stresses,

σ_d = the applied deviator stress, and

k_1, k_2, k_3 = regression constants.

Among these coefficients, the k_1 is selected as the best

Table 6. Coefficients of the universal soil model for the base and subgrade materials

Layer	State	SHRP ID	k_1 (kPa)	k_2	k_3	Moisture Content (%)
Base	GA (13)	1005	829.1	0.669	−0.147	6.8
	GA (13)	1031	627.0	0.677	−0.248	6.1
	MN (27)	6251	743.3	0.710	−0.031	5
	MS (28)	1016	631.8	0.670	−0.119	10.4
	MS (28)	1802	1030.5	0.528	−0.140	6.5
	TX (48)	1068	585.2	0.699	−0.021	6.7
	TX (48)	1077	1070.7	0.660	−0.038	6
Subgrade	AL (1)	0101	555.6	0.19	−0.17	19.9
	AL (1)	0102	650.0	0.16	−0.14	21.6
	GA (13)	1005	520.9	0.47	−0.09	12.5
	GA (13)	1031	255.3	0.57	−0.27	10.9
	MN (27)	6251	268.3	0.57	−0.23	7.4
	TX (48)	1060	329.1	0.36	−0.30	21.5
	TX (48)	1068	370.3	0.04	−0.21	18.8
	TX (48)	1077	515.9	0.42	−0.14	10.7

indicator for representing the stiffness characteristic of the base and subgrade layers. Park and Fernando (1998) also concluded that the k_1 is the most influential coefficient for the resilient modulus of unbound materials. For the base layer condition assessment, the BDI and DBDI values adjusted using the structural correction procedure were plotted against the k_1 values of the base materials in Figure 4. Although there are possible errors due to the location of testing-specific variations in the stiffness of pavement materials, it is observed that the k_1 value decreases with an increase in the adjusted BDI and DBDI values with relatively good correlations.

Figure 5 presents the relationships between the adjusted ε_{abc} and $d\varepsilon_{abc}$ values, and the k_1 values of the base materials. It can be seen that the k_1 decreases as the ε_{abc} and $d\varepsilon_{abc}$ increase. The ε_{abc} and $d\varepsilon_{abc}$ result in a slightly lower degree of correlation than the BDI and DBDI. This trend may be due to the fact that there is an additional step of prediction (i.e., from deflections to pavement responses) involved in using the compressive strain as a condition indicator than using the deflection basin parameters directly. This additional step causes more approximation and errors and therefore poorer correlations with actual layer condition. It is also noted by comparing R^2 values in Figure 5 that the $d\varepsilon_{abc}$ values, and therefore the use of multi-load deflections, negatively impact the correlation between the pavement response and k_1 para meter.

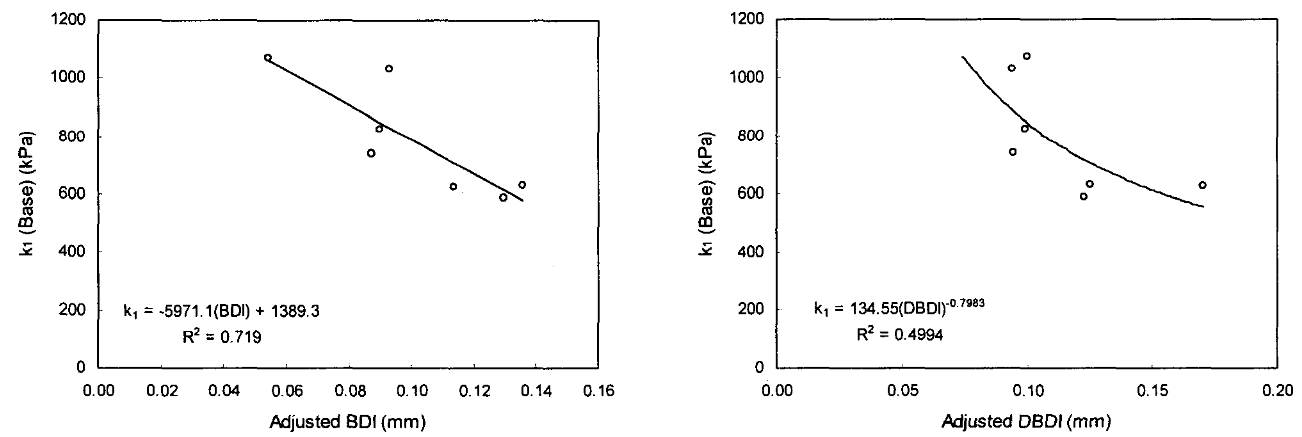


Fig. 4. Adjusted BDI and DBDI versus k_1 of aggregate base for LTPP test sections

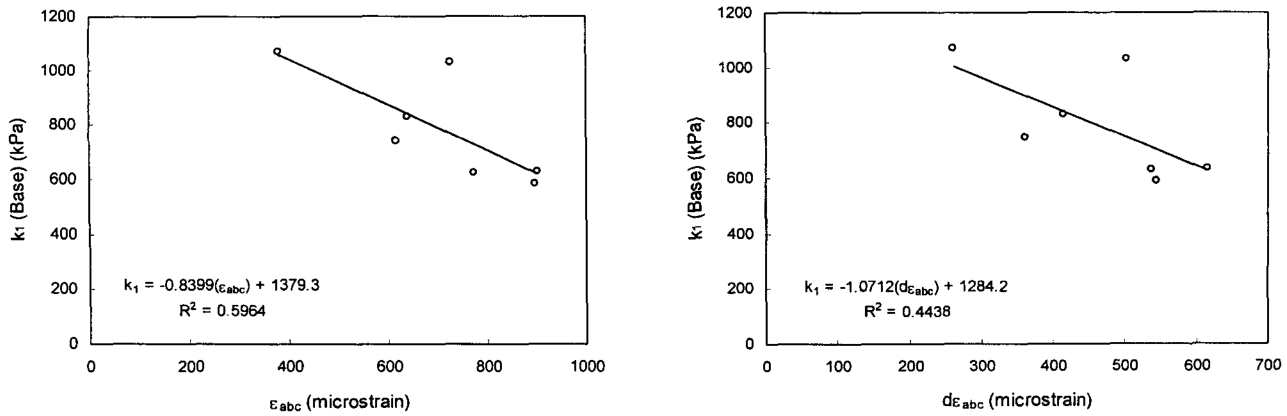


Fig. 5. Adjusted ε_{abc} and $d\varepsilon_{abc}$ versus k_1 of aggregate base for LTPP test sections

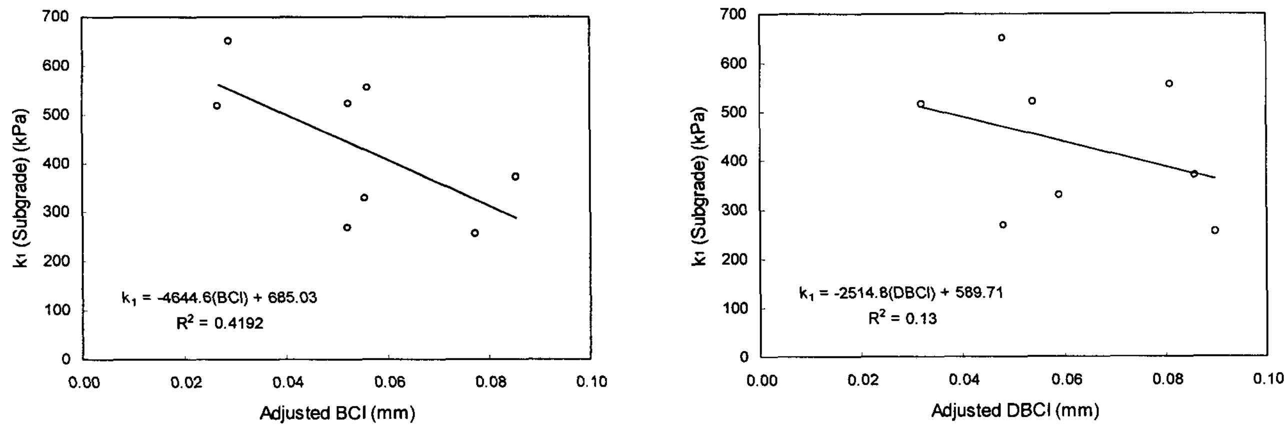


Fig. 6. Adjusted BCI and DBCI versus k_1 of subgrade for LTPP test sections

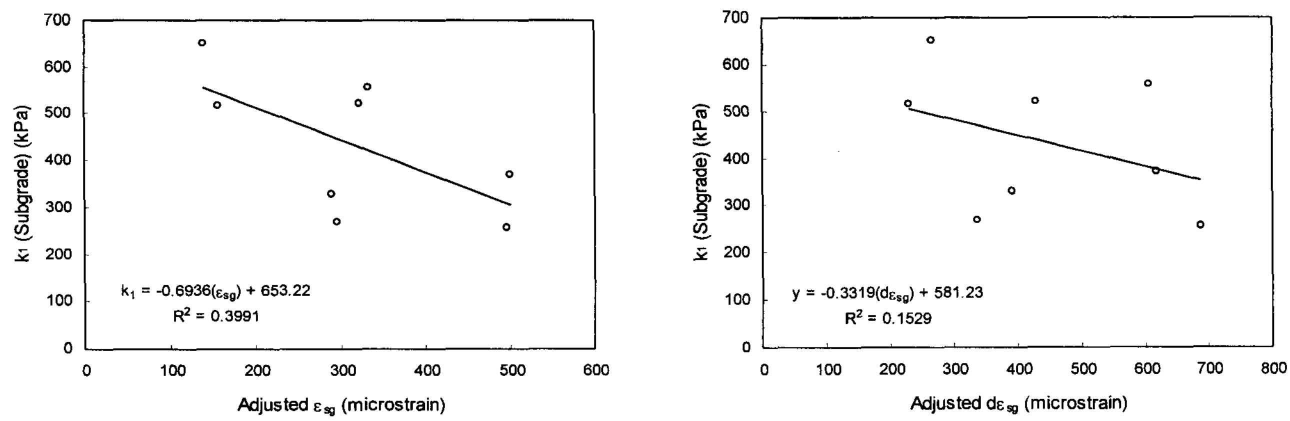


Fig. 7. Adjusted ε_{sg} and $d\varepsilon_{sg}$ versus k_1 of subgrade for LTPP test sections

The BCI and DBCI values are plotted in Figure 6 against the k_1 values of the subgrade materials. It was found from Figure 6 that the k_1 decreases as the adjusted BCI increases with a reasonable degree of correlation. However, the adjusted DBCI shows a much larger variation and may not be suitable for an indicator for the condition of subgrade. Figure 7 shows the adjusted ε_{sg} and $d\varepsilon_{sg}$ values versus the k_1 values for the subgrade soils respectively. It can be found that the degrees of correlation using the subgrade strain are close to those using the deflection basin parameters. In summary, the following conclusions can be made from Figures 4 to 7:

1. Structurally adjusted BDI and BCI are good indicators for the prediction of k_1 values of aggregate base and

subgrade respectively;

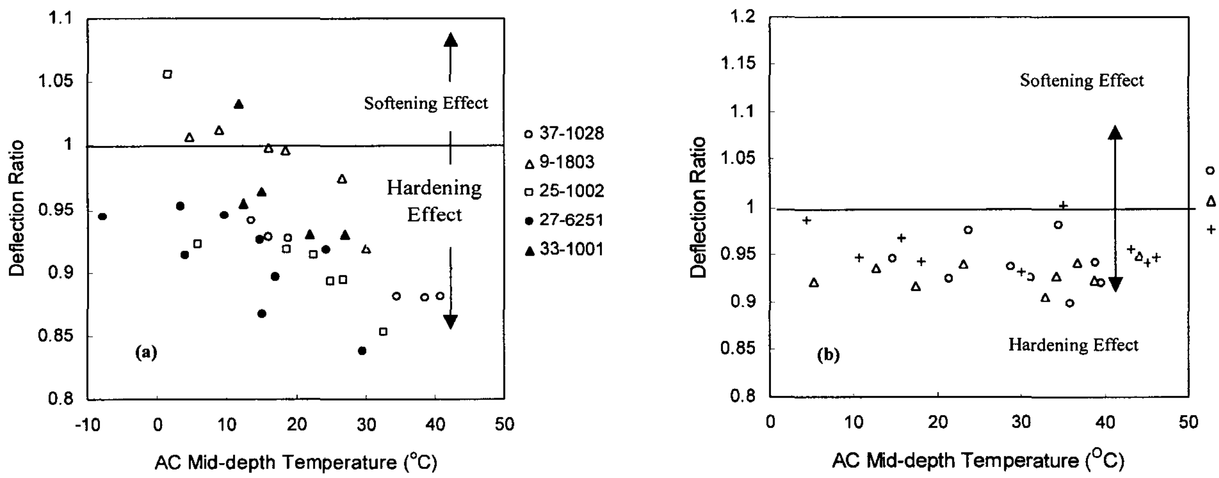
2. In general, the degree of correlation between k_1 and the deflection basin parameters is better than that with the compressive strain;
3. The use of multi-load deflections does not improve the prediction accuracy for the k_1 values of aggregate base and subgrade layers.
4. The same type of investigation was conducted using the k_2 and k_3 parameters of aggregate base and subgrade materials and revealed that there exists no reliable relationship between condition indicators and k_2 and k_3 parameters.

7. Effect of Load Level on the Nonlinear Behavior of a Pavement Structure

To check the nonlinearities for flexible pavements, measured surface deflections were normalized with respect to a load level. To determine the degree of nonlinearity in a pavement structure, deflection ratios can be calculated by dividing the normalized deflections under a 71.2 kN load by the normalized deflections under a 26.7 kN load (Chang, 1991).

The deflection ratio concept was applied to the FWD deflections obtained from test sections in DataPave 2.0. The deflection ratios are plotted against the AC mid-depth temperatures in Figure 8 for pavements with gravel, and crushed stone base layer. It is noted that the subgrade soils in these pavement sections are silty or granular sandy materials except for the 48-1068 section. For gravel and crushed stone base pavements, the deflection ratios are less than one at a wide range of temperatures, which demonstrates the possible hardening behavior of pavement materials. It is well known that as the AC mid-depth temperature increases, the AC modulus decreases, and then stress in the base and subgrade layers increases simultaneously. The modulus of granular materials increases as the stress increases (the hardening effect), whereas the reverse trend is observed in fine-grained soils (the softening effect). As shown in Figure 8, overall the deflection ratio of the gravel base pavements decreases as the AC mid-depth temperature increases. This trend can be explained by the well-known effect of bulk stress on the modulus of granular materials. However, the deflection ratios for crushed stone base pavements were relatively constant, regardless of the AC mid-depth temperature and subgrade soil type. This trend is because the modulus of crushed stone is very high and seems to be less sensitive to change in stresses.

Further investigation was conducted to determine the effect of subgrade soil type on the nonlinear behavior of a pavement structure. As shown in Table 4, the subgrade soils in the 31-0114 and 1-0102 sections are classified as CL, indicating a plastic clayey material, whereas the subgrade soils in the 25-1002 and 27-6251 sections are SP, which is a granular sandy material. It should be noted



(a) (b)

Fig. 8. Deflection ratio versus AC mid-depth temperature for pavements with: (a) gravel base layer (b) a crushed stone base layer

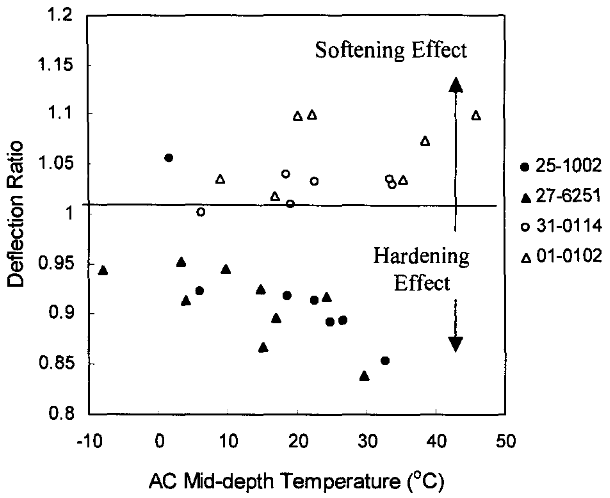


Fig. 9. Effect of subgrade soil type on nonlinear behavior of a pavement structure

that the thickness of the AC and base layers and the type of base materials are almost the same in these sections. It is observed from Figure 9 that the deflection ratios in pavements with a CL soil are larger than one and increase with increasing AC mid-depth temperatures, while the deflection ratios in pavements with a SP soil are less than one and decrease with increasing AC mid-depth temperatures. This study concludes that the deflection ratio is a very useful parameter to predict the soil type in the subgrade layer.

8. Conclusions

This research effort focused on the development of procedure of the pavement condition assessment using multi-load level FWD deflections. The measured FWD multi-load level deflections, DCP data, and resilient modulus test results for foundation materials were used to validate the condition assessment procedure. The BDI and DBDI are capable of estimating the strength of subgrade materials in full depth pavements. It is also found that structurally adjusted BDI and BCI are good indicators for the prediction of k_1 values of aggregate base and subgrade respectively. A 66.7 kN of FWD load level seems not large enough to improve the accuracy in assessing base and subgrade layer condition.

Results from the study for nonlinear behavior of a pavement structure indicate that the deflection ratio obtained from multi-load level deflections can predict the type and quality of base/subgrade materials.

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