Nonlinear Effects in Falling Weight Deflectometer Tests

DER-WEN CHANG, JOSE M. ROESSET, AND KENNETH H. STOKOE II

The falling weight deflectometer (FWD) has been used in the evaluation of material properties of pavement systems for many years. The load amplitude and frequency content are intended to provide deformation levels similar to those induced by truck wheel loads. Interpretation of the in situ measured data is normally based on an elastic solution and therefore does not take into account the possible existence of localized nonlinearities. The objective of this work was to develop some understanding of the potential for nonlinear behavior in the FWD test. Analytical studies were conducted using both a linear iterative solution and a nonlinear solution with the generalized cap model to reproduce the nonlinear soil behavior. Three pavement sections in Texas were considered using the finite discrete model. By varying the level of loads FWD deflection basins, induced strains, and inelastic material properties were obtained. The results of this study indicate that material nonlinearities are localized and are important for FWD tests on flexible pavements where the subgrade is relatively soft or the pavement is thin. In these cases, nonlinear effects increase peak displacements by at least 50 percent under a 20,000-lb load but are negligible for receivers at 3 ft or more from the source. In general, a FWD test with a load of 2,000 lb or less would not result in any apparent nonlinear effects at any pavement site. For FWD tests on rigid pavements or flexible pavements with a relatively stiff subgrade, nonlinearities are also less pronounced.

Nondestructive testing techniques have been used by highway engineers for years to evaluate in the field the structural capacity and integrity of existing pavements. Among a number of available testing methods, the falling weight deflectometer (FWD) is commonly considered to provide estimates of material properties for levels of load similar to those exerted by truck wheels as discussed by Uddin et al. (*1*). In this method, a hydraulically lifted weight is dropped on top of a disc mounted on the surface of the pavement and outward by propagating wave motions are recorded by a set of velocity transducers placed on the surface (Figure 1). The peak displacements of these receivers form a deflection basin that is then used to backfigure the material profile on the basis of elastic multilayer theory. In general, the peak of the FWD load ranges from 1,500 to 24,000 lb with frequencies between 0 and 60 Hz. An illustration of the idealized FWD loading function used for the present study is shown in Figure 2.

A number of research studies have been conducted recently to understand the effect of various factors on the FWD measurements such as the presence of much stiffer bedrock at a finite depth. Roesset and Shao (*2*), Davies and Mamlouk (*3*), and Chang et al. in another paper in this Record have concentrated on the importance of dynamic effects. Kang et al. (*4*) investigated the effect of the finite width of the pavement. Dynamic wave phenomena were found to be important in some cases in which the effects ought to be taken into account to ensure appropriate backcalculations. Another concern in analyzing the measured data is the possible nonlinear behavior of the paving materials induced by large load levels. Such effects were reported by Nazarian and Stokoe (*5*), who performed a set of FWD measurements on a secondary road. Although the occurrence of nonlinear behavior has been considered an advantage in better simulating the levels of strain and stress caused by the truck wheels, it is necessary to account for the magnitude and spatial distribution of these nonlinearities under different loads for a proper interpretation of the data. The purpose of this work is to estimate the degree of nonlinearity that can be expected in different types of pavements as a function of the load magnitude, and to estimate associated errors in backfiguring the material moduli. The work is limited to analytical predictions intended to provide an expected order to magnitude. Experimental work is needed to validate these predictions.

D.-W. Chang, Department of Civil Engineering, Tamkang University, 180 Chung-An Street, Chung-Ho City, Taipei Hsien, Republic of China. J. M. Roesset and K. H. Stokoe II, Department of Civil Engineering, University of Texas at Austin, Austin, Tex. 78712–1076.

NONLINEAR CHARACTERISTICS OF THE PAVING MATERIALS

The most often used materials for the pavement surface layer are portland cement concrete and asphalt concrete. The load-dependent behavior of these materials is not of interest here because they are considerably stiffer than the base and subgrade materials (soils). Nevertheless, it is necessary to point out that loading rate would affect the dynamic modulus of the asphalt mixtures, particularly for high temperatures.

For the base, the focus of interest in the present study is on untreated granular materials. Deformational characteristics of granular materials depend on the strain level and the state of stress. Such relations are commonly expressed in highway engineering in terms of the resilient modulus and 1-D recoverable strain. Alternatively, they can be represented as the variation of shear modulus and damping with shear strain as normally done in earthquake engineering. Many factors have been found to affect the shear modulus (G) and damping ratio of soils. Hardin and Drnevich (*6*) ranked several factors in order of their importance including state of stress, void ratio and strain amplitude. For a given material with known void ratio and state of stress, the nonlinear behavior is normally represented by plotting the modulus and damping versus shear strain. As a general rule of thumb, material response

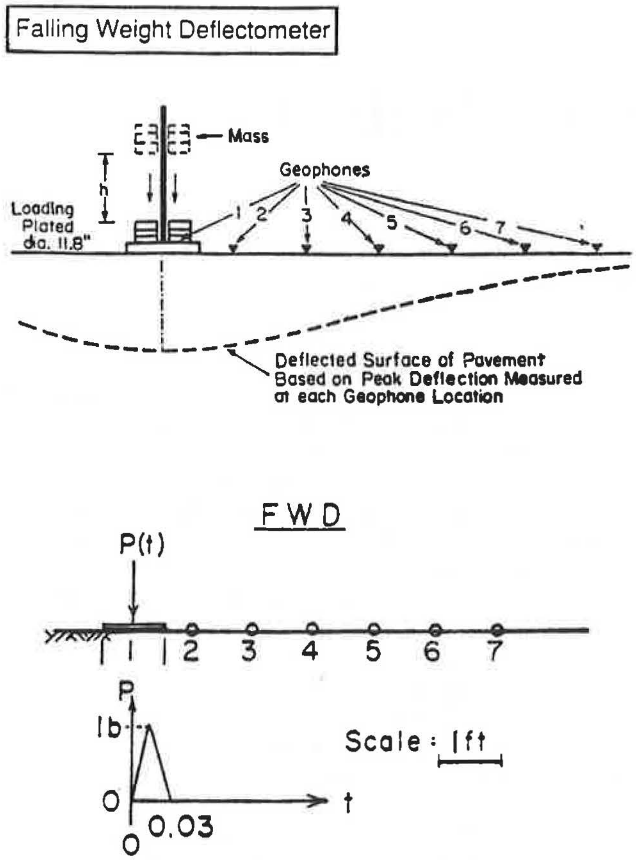

FIGURE 1 Layout of the in situ instrumentation and geometric configuration of the load and stations of FWD test (*1*).

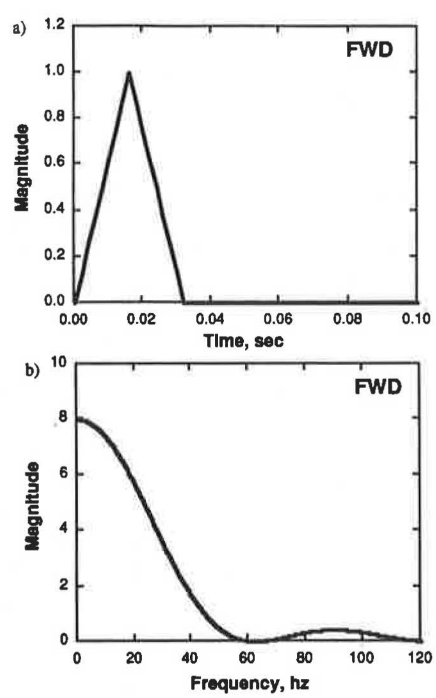

FIGURE 2 Simplified loading history of FWD test and corresponding load spectra.

is considered to be linear elastic for strain levels less than 0.001 percent, nonlinear elastic (reversible load-displacement relations) for shear strains between 0.001 percent and 0.01 percent, and inelastic for strains larger than 0.01 percent. A comprehensive review of the research on shearing characteristics of soils can be found in work by Ni (*7*).

The nonlinear material characteristics used in this study are based on laboratory data on clayey and sandy soils. Both rely on samples representing common materials collected at pavement sites in Texas. A typical plot of the variation of the shear modulus and damping with shear strain amplitude is shown in Figure 3. To simplify the material profiles at different sites, the curves for sandy soils are considered as representative of granular bases, and the data of clays are used to model the subgrade. Owing to the lack of more complete laboratory data on asphalt concrete, strain amplitude and rate dependence of its shear modulus and damping are neglected.

ANALYTICAL MODEL AND APPROACHES

Determination of the displacements, strains, and stresses created in a layered medium by dynamic loads has been a subject of considerable interest and research for the last 20 years in relation to the design of machine foundations and the seismic analyses of buildings and foundations. A number of procedures have been developed to solve the linear problem by modeling the ground with a discrete model and an absorbing boundary to reproduce the far-field or by applying analytical solutions with a Fourier series expansion in two or three dimensions. For the nonlinear problem, a model that has a core

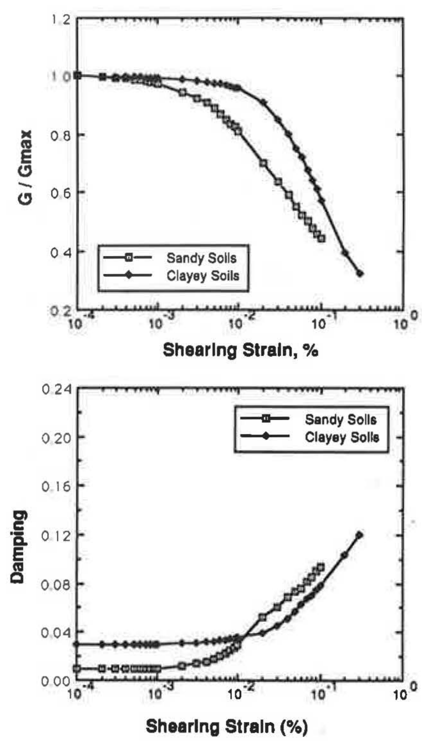

FIGURE 3 Nonlinear shear deformational characteristics of subgrade soils.

region discretized with finite elements and an appropriate set of boundary conditions at a finite distance is more attractive. Nonlinear analyses related to foundation engineering have been conducted by Lysmer et al. (*8*) and Roesset and Scaletti (*9*) assuming linear hysteretic soil properties, accounting approximately for the nonlinear soil behavior using the so-called linear iterative or equivalent linear approach. In this work, dynamic analyses were conducted using two approaches: the approximate nonlinear approach and a true nonlinear analysis.

The first approach, the linear iterative procedure, is conducted in the frequency domain, using prescribed nonlinear curves of modulus and damping versus strain to select in each cycle properties consistent with the levels of strain computed in the previous one; the time histories of the solutions are then obtained using fast Fourier transform. Corrections of the material properties are based on an equivalent measure of shear strain defined as the shearing strain intensity Γ = [$\Gamma = (2\bar{J}_2)^{1/2}$ where $\bar{J}_2$ is the second invariant of the shear strain tensor]. A lateral absorbing boundary is placed at an appropriate distance from the source to reproduce the energy dissipation in the far field as suggested by Kausel et al. (*10*).

Using the second approach, a true nonlinear analysis, the solution is obtained in the time domain integrating the equations of motion of the system. A generalized cap model proposed by Sandler et al. (*11*) is used to reproduce the nonlinear material behavior (Figure 4). The equations used in the model for failure envelope, hardening cap and the nonlinear moduli suggested by Chen and Baladi (*12*) are discussed in detail by Chang (*13*).

Because the data necessary to fit the parameters of the plasticity model were not available, no attempt was made to reproduce exactly the actual behavior of a specific site. The objective of the study was, instead, to get an idea of the degree of nonlinearity that could be expected using two different models with consistent values.

The discrete models used for both types of analyses are shown in Figure 5. The details of the formulation, computational schemes, model behavior, and simulation as well as material correlations for the nonlinear analyses can be found in work by Chang (*13*).

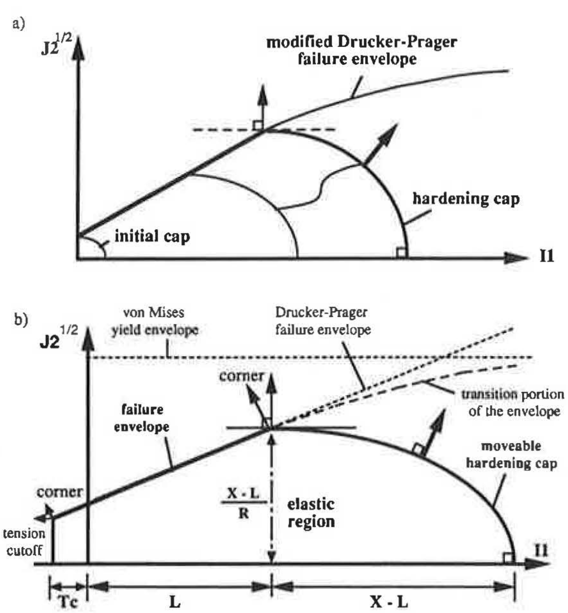

FIGURE 4 Configuration of the cap model in the I1–J2 space.

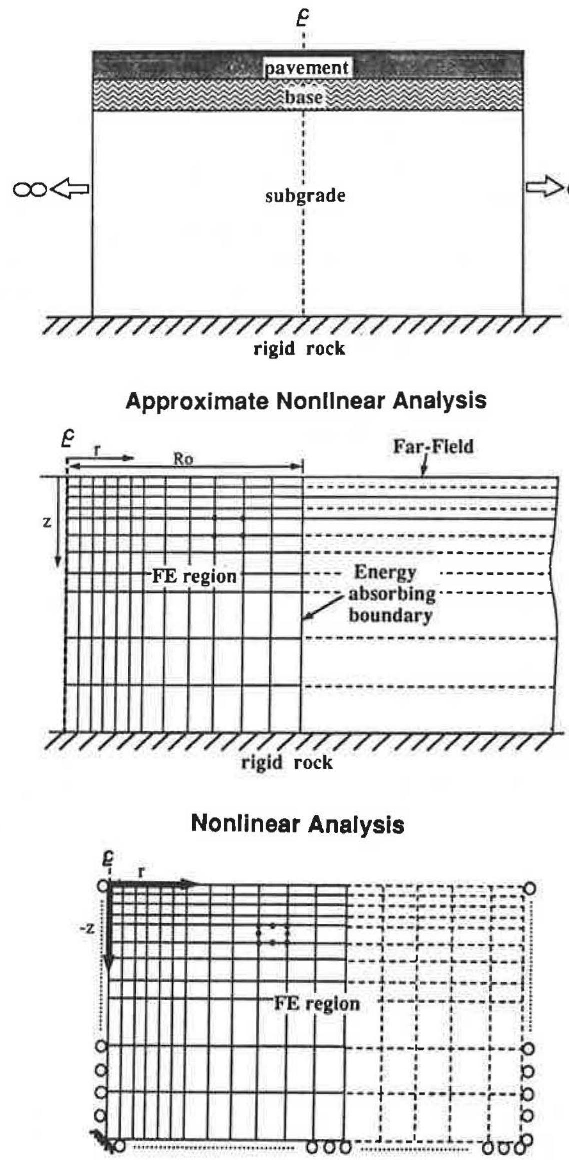

FIGURE 5 Simplified pavement systems and discrete models for analyses.

CASE STUDIES

Site Description

To investigate the possible existence of nonlinearities and their localization at various typical pavement sites, three pavement sections were selected as material profiles for analysis. They correspond to farm-to-market road FM 195 in Paris, Texas; Route 1 in Austin, Texas; and Interstate 10 in El Paso, Texas. Road FM 195 was selected to represent a typical flexible pavement on relatively soft subgrade. Route 1 represents a flexible pavement where the subgrade is relatively stiff. Interstate highway 10 represents a typical rigid pavement. The elastic material properties and layer thicknesses of each profile as reported to the authors from measurements are presented in Table 1. Rigid rock was placed at a depth of 20 ft in all cases to reduce the size of the finite element mesh and to save computational time. Clearly the depth to bedrock influences the dynamic amplification and therefore the strains.

The material parameters in the nonlinear gap model were selected to yield variation of modulus and damping curves similar to those used in the approximate approach. The values were reported by Chang (*13*). To generalize the initial state, a hydrostatic stress of 10 psi was preassigned to the soils in order to place the initial cap.

Approximate Nonlinear Analyses

Deflection Basin

The results of the deflection basin normalized with respect to the load (deflections per pound) for 1, 1,000, 2,000, 5,000, 10,000 and 20,000-lb loads and the corresponding deflection ratios with respect to the 1-lb load are plotted in Figures 6 and 7 for the approximate nonlinear analysis. Important nonlinearities on the peak displacements are found at the first two receivers (which are within 2 ft from the source) in site FM 195 for loads over 5,000 lb. These effects would cause an increase of approximately 56 percent in the peak displacements at site FM 195 with a 20,000-lb load. For Route 1 and I-10, nonlinear effects were found to be insignificant for all load levels.

Variation of Nonlinear Material Moduli and Damping

Variations of the material shear moduli (in the normalized form) and the damping using the 20,000-lb load at site FM 195 are presented in Tables 2 and 3. Note that the shear modulus of the base layer can be reduced to about 14 percent of its original value. The shear modulus in the upper portion of the subgrade layer decreases to 45 percent of its initial value. The associated maximum material damping at the site is 11 percent in the base layer and 9 percent in the upper subgrade layer. The localized nature of these nonlinearities are clearly illustrated and an assumption of lateral homogeneity, as is implicitly done in normal backcalculation procedures, is not appropriate.

TABLE 1 Material Properties and Thickness of Profile at Sites

Layer	Thickness (ft)	Vs (fps)	υ	E (ksi)	γ (pcf)
FM 195					
ACP	0.333	2500	0.15	434	140
Base	0.50	800	0.25	43	125
Subgrade	19.167	500	0.33	16	110
Route 1					
ACP	0.583	2000	0.27	318	145
Base	0.50	1000	0.25	76	130
Subgrade	18.917	1000	0.33	76	130
IH 10					
CRC	0.833	8500	0.20	5423	145
AC	0.5	2000	0.27	318	145
Base	1.0	800	0.33	46	125
Subgrade	17.667	500	0.33	18	125

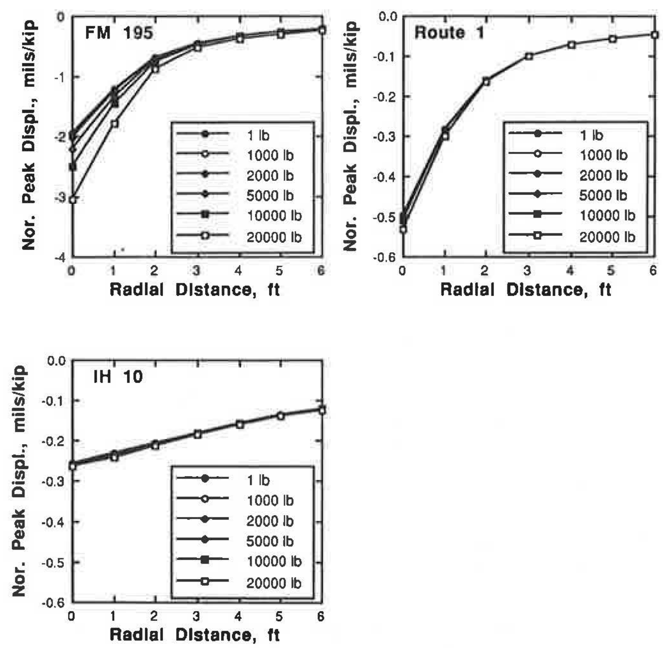

FIGURE 6 Dynamic deflection basins at various sites from approximate nonlinear analysis.

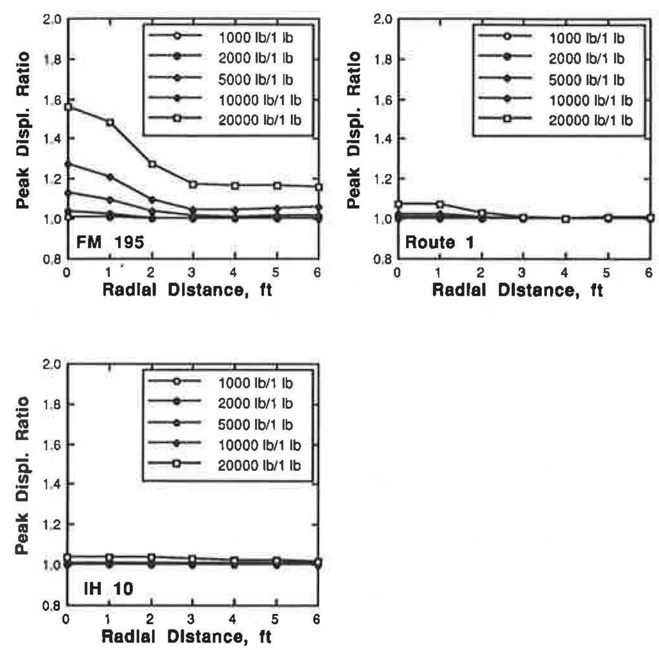

FIGURE 7 Ratio of deflection basin at various sites from approximate nonlinear analysis.

TABLE 2 Variation of G/G_{max} for FM 195 with 20,000-lb FWD Load

	Radial Distance, ft												
Depth, ft	.25	.75	1.25	1.75	2.25	2.75	3.25	3.75	4.25	4.75	5.25	5.75	6.25
.167	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
.583	.138	.162	.193	.232	.277	.326	.374	.420	.475	.521	.560	.596	.632
1.083	.451	.519	.590	.645	.726	.777	.819	.859	.897	.906	.913	.919	.925
1.583	.500	.568	.615	.684	.752	.796	.837	.876	.903	.909	.915	.921	.928
2.083	.541	.594	.646	.722	.773	.815	.856	.895	.906	.913	.919	.925	.931
2.583	.574	.614	.680	.750	.793	.835	.876	.903	.910	.916	.922	.928	.935
3.083	.590	.634	.711	.769	.812	.856	.897	.907	.914	.920	.926	.932	.938

TABLE 3 Variation of Material Damping for FM 195 with 20,000-lb FWD Load

	Radial Distance, ft												
Depth, ft	.25	.75	1.25	1.75	2.25	2.75	3.25	3.75	4.25	4.75	5.25	5.75	6.25
.167	.020	.020	.020	.020	.020	.020	.020	.020	.020	.020	.020	.020	.020
.583	.110	.104	.096	.088	.080	.075	.071	.067	.063	.059	.056	.054	.051
1.083	.089	.083	.076	.070	.064	.057	.052	.049	.045	.043	.041	.039	.038
1.583	.085	.079	.073	.067	.061	.055	.050	.047	.044	.042	.040	.039	.038
2.083	.081	.076	.070	.064	.058	.053	.049	.045	.043	.041	.039	.039	.038
2.583	.079	.073	.068	.062	.056	.051	.047	.044	.042	.040	.039	.038	.038
3.083	.076	.070	.065	.059	.053	.049	.045	.043	.041	.039	.038	.038	.037

Nonlinear Analyses

Deflection Basin

The normalized deflection basins and their ratios are plotted in Figures 8 and 9 for the nonlinear analyses. In the case of FM 195, the nonlinear effects of a 20,000-lb load cause an increase of about 140 percent in the peak displacement at the first receiver. For Route 1, the same load results in an increase of the displacement of about 50 percent, much more significant than the increase predicted by the previous solution. In general, the nonlinearities for relatively large loads (> 10,000 lb) are important within 3 ft from the source at these two sites. The displacements recorded at the farther receivers become smaller with increase in the load. For site I-10, nonlinear effects were again found. The true nonlinear displacement was about 50 percent larger than the approximate one (from 1.6 to 2.4 times the linear displacement at FM 195). This was expected and has been reported in the past.

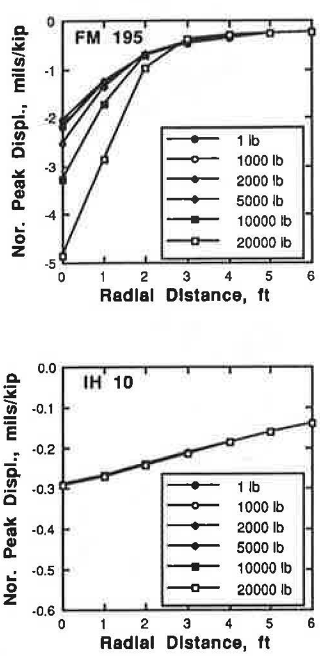

FIGURE 8 Dynamic deflection basins at various sites from nonlinear analysis.

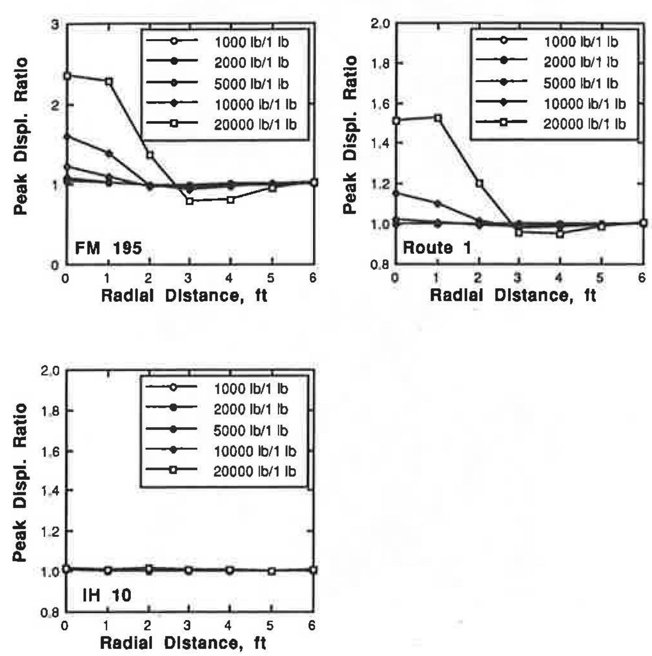

FIGURE 9 Ratio of dynamic deflection basins at various sites from nonlinear analysis.

Variation of Linear and Nonlinear Strain Field

The variation of the shear strains underneath the pavements using 1 lb and 20,000 lb loads at all sites is plotted in Figures 10 – 12. The maximum strain is always found in the base layer. The levels of the strain distribution can be used to correlate the variation of the material properties that induce the inelastic phenomena.

CONCLUSIONS

Dynamic nondestructive tests for evaluating the structural capacity of pavements have become a popular tool for highway engineers in selecting rehabilitation and reconstruction strategies. A considerable amount of research has been conducted in recent years to improve the understanding of the factors affecting these tests from both the analytical and experimental points of view. Although some problems still have not yet been fully resolved specially in the interpretation of the measured data, a distinct nonlinear phenomenon (load dependence) has often been noticed in FWD measurements on light pavements. In this study investigations based on forward analyses modeling the nonlinear phenomena were conducted to predict the response of the pavements. A study was first conducted using an approximate nonlinear solution. Two sets of experimental nonlinear curves were used to model the

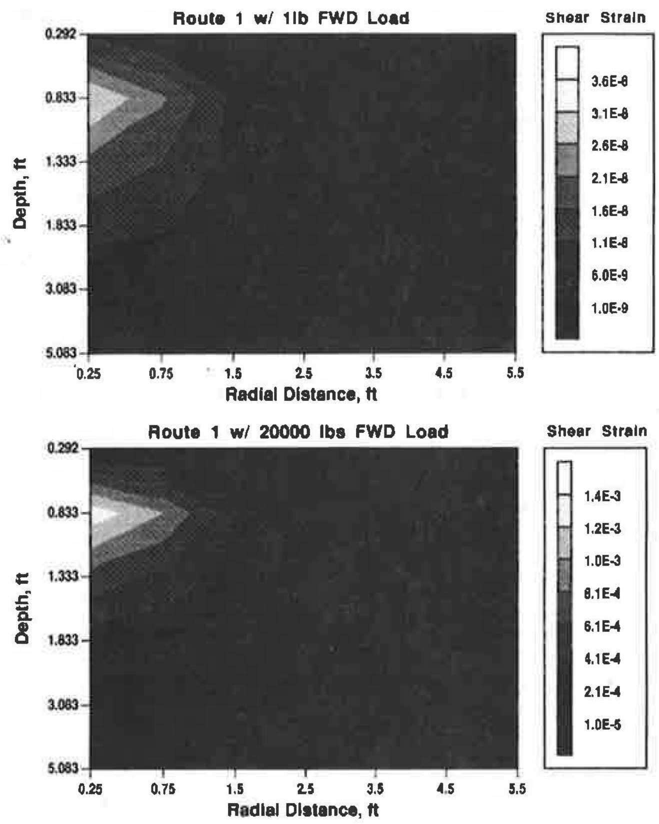

FIGURE 11 Variation of shear strain at Route 1 using 116 and 20,000-lb FWD loads.

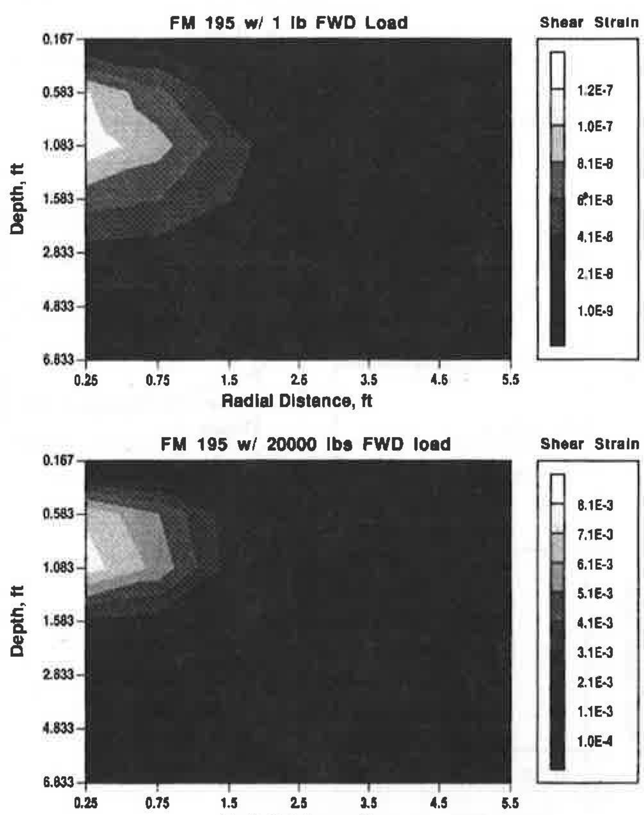

FIGURE 10 Variation of shear strain at FM 195 using 116 and 20,000-lb FWD loads.

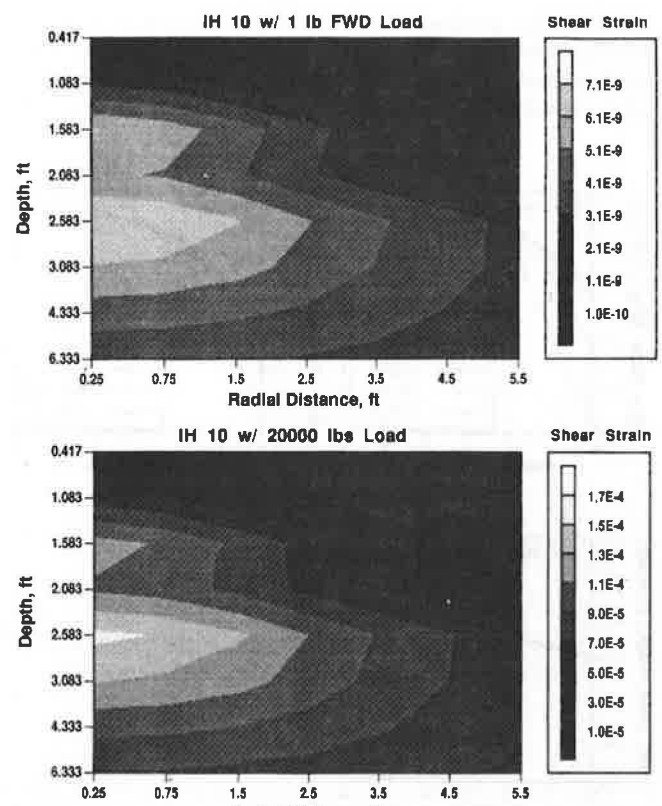

FIGURE 12 Variations of shear strain at I-10 using 116 and 20,000-lb FWD loads.

shear deformational characteristics of the base and the subgrade materials. To simulate more realistically nonlinear response, analyses were also conducted integrating the equations of motion in time with the generalized cap model to reproduce material behavior. A set of analyses was conducted for three pavement sites using both methods. Solutions in terms of surface displacements, strain distribution and the modulus and damping spread were presented.

The results of these studies indicate that nonlinear effects on the measured deflections are directly related to (a) the magnitude of the load, (b) the type of pavement, (c) the stiffness of the subgrade, and (d) the thickness of the pavement surface. If the test is performed on a flexible pavement, a thin surface, a relatively soft subgrade, and with a large value of the load, large nonlinearities will take place near the source.

Results also indicate that for the FWD measurements at farm-to-market roads such as FM 195 where the nonlinearities are important, the maximum difference in peak displacements due to the nonlinear effects of a 20,000 lb load can be at least 50 percent. The nonlinear effects will be smaller for receivers at distances beyond 3 ft from the source.

The nonlinear effects obtained from the nonlinear analyses are greater than those from the approximate analyses. To resolve this problem more carefully, complete information of the material behavior under proper loading (using proper testing) must be obtained to calibrate the parameters used for the nonlinear material model.

The possible effects of material nonlinearities on FWD measurements were studied analytically in this work. Correlation of these results with field measurements is necessary. The approximate nonlinear analysis provides a better qualitative understanding of the material variations under the road. It also requires less data on the base and subgrade properties. It may underestimate strains, however, by about 50 percent. The true nonlinear analysis would provide more accurate results if the nonlinear constitutive equations were realistic and their parameters were known for the material in question. To fit these parameters would require a considerable amount of sample collection and laboratory testing. This type of analysis is thus more suited for research purposes than for actual use in practice on a regular basis. Both methods revealed the potential for nonlinearities using large FWD loads particularly for flexible pavements and soft subgrades. The question that must still be resolved is the meaning of the backfigured moduli when the properties change in the horizontal direction as will happen when there is nonlinear behavior. Both methods revealed the potential for nonlinearities using large FWD loads. At arbitrary sites, the degree of the nonlinear effects is indeed related to the actual thickness of the soil stratum and should be studied independently.

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