

# Response of Pavement Systems to Dynamic Loads Imposed by Nondestructive Tests

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Dynamic nondestructive testing of pavements can be grouped into two categories: (a) deflection basin tests and (b) wave propagation tests. Among the deflection basin tests, the Falling Weight Deflectometer test (FWD) has seen the most widespread use. Of the wave propagation tests, the Spectral Analysis of Surface Waves (SASW) test can be used with or serve as an alternative to deflection basin tests. The theoretical formulation used to analyze the dynamic response of pavement systems to dynamic loads imposed by such nondestructive testing techniques is presented. Analytical studies of the dynamic response of two generalized pavement systems (a flexible one and a rigid one) to the FWD and SASW tests were carried out. The results indicate that dynamic deflection basins due to the FWD load can be substantially different from those obtained under static conditions. The study shows that when complete time histories of FWD deflections are stored, the additional information can provide substantial insight into the properties of the pavement system and can significantly facilitate the inversion process. The dispersion curves obtained with the SASW test are very sensitive to the stiffness of the surface and subgrade layers but, unfortunately, are relatively insensitive to the properties of the base layer. These results are true even when bedrock is present at shallow depths.

Dynamic nondestructive testing techniques have been used for years to evaluate the structural capacity and integrity of highway and airfield pavements. These techniques can be grouped into two categories: (a) deflection basin tests and (b) wave propagation tests. Deflection basin tests are those in which maximum deflections are recorded along the surface of a pavement subjected to a steady-state harmonic load or a transient dynamic load. Typical of this group are the Dynaflect and Road Rater tests (steady-state loads) and the Falling Weight Deflectometer test (transient load) [Uddin (1)]. At present, interpretation of deflection basins, in order to backfigure the moduli of the pavement layers, is normally performed with static analyses and assuming that the subgrade extends to infinity. This approach neglects the dynamic nature of these tests and the fact that, in many cases, the soil will be underlain at some depth by much stiffer, rocklike material.

The second category of dynamic nondestructive tests is wave propagation tests. In these tests, the time histories of motion due to an applied dynamic load are recorded at two or more receivers on the pavement surface. By computing the surface wave travel time between adjacent receivers for different excitation frequencies, a dispersion curve is obtained relating phase velocities to frequencies (or wavelengths). Thicknesses and stiffnesses of the pavement layers are then obtained by an inversion process based on the propagation of generalized plane surface waves of the Rayleigh type. This type of test is represented by the Spectral Analysis of Surface Waves (SASW) method [Nazarian and Stokoe (2)].

Among these testing techniques, the Falling Weight Deflectometer (FWD) test has seen the most widespread use, largely because

of its ability to impose on a pavement high-amplitude dynamic loads similar to those imposed by truck traffic. The FWD consists of a drop weight mounted on a vertical shaft and housed in a trailer that can be towed by most conventional vehicles. The drop weight is hydraulically lifted to predetermined heights ranging from 5 to 51 cm (2 to 20 in.). The weight is dropped onto a 30-cm-(11.8-in.-)in diameter loading plate resting on a 5.6-mm-(0.22-in.-)thick rubber buffer. The resulting load is a force impulse with a duration of approximately 30 msec and a peak magnitude ranging from about 9,000 to more than 90,000 N (2,000 to 20,000 lb), depending on the drop height and drop weight. The force and deflection signals at various points along the surface are measured by a load cell and a set of vertical velocity transducers. The position of the load and recording stations are shown in Figure 1(a).

In the SASW test, an impulsive load is applied at the surface, and the passage of the surface wave train is monitored at two vertical receivers, also located on the surface. A spectral analysis of the two recorded signals is performed, and the variation of the surface wave velocity with frequency or wavelength (dispersion curve) is found. The general arrangement of the source, receivers, and recording equipment is shown schematically in Figure 1(b). A piezoelectric shaker can be used to generate surface waves ranging from about 1 kHz to 50 kHz. The high frequencies are necessary to sample the stiff surface layer. A digital waveform analyzer, coupled with a microcomputer, is used to capture and process the output from the receivers. Compared with the FWD test, the SASW test imposes smaller loads on the pavement surface, but data analysis is more complex, and automation is less well-developed in the SASW test than in the FWD test.

An analytical study of the dynamic response of pavement systems to forces simulating the excitations of the FWD and SASW tests is presented. The FWD deflection basins obtained with a dynamic model and those computed with static analyses are compared. The effects of the depth to bedrock on the deflection basins (FWD) and on the dispersion curves (SASW) are investigated. The sensitivity of both types of tests to the properties and thicknesses of the various layers is also discussed.

## THEORETICAL FORMULATION

To understand how a pavement system responds to dynamic loads applied to the surface, a review of the theoretical studies that deal with the dynamic response of uniform and layered systems is necessary.

### Dynamic Loads on a Semi-Infinite Medium

Lamb (3) was the first to study the effect of a pulse on a uniform elastic half-space. Lamb treated four basic problems: surface line

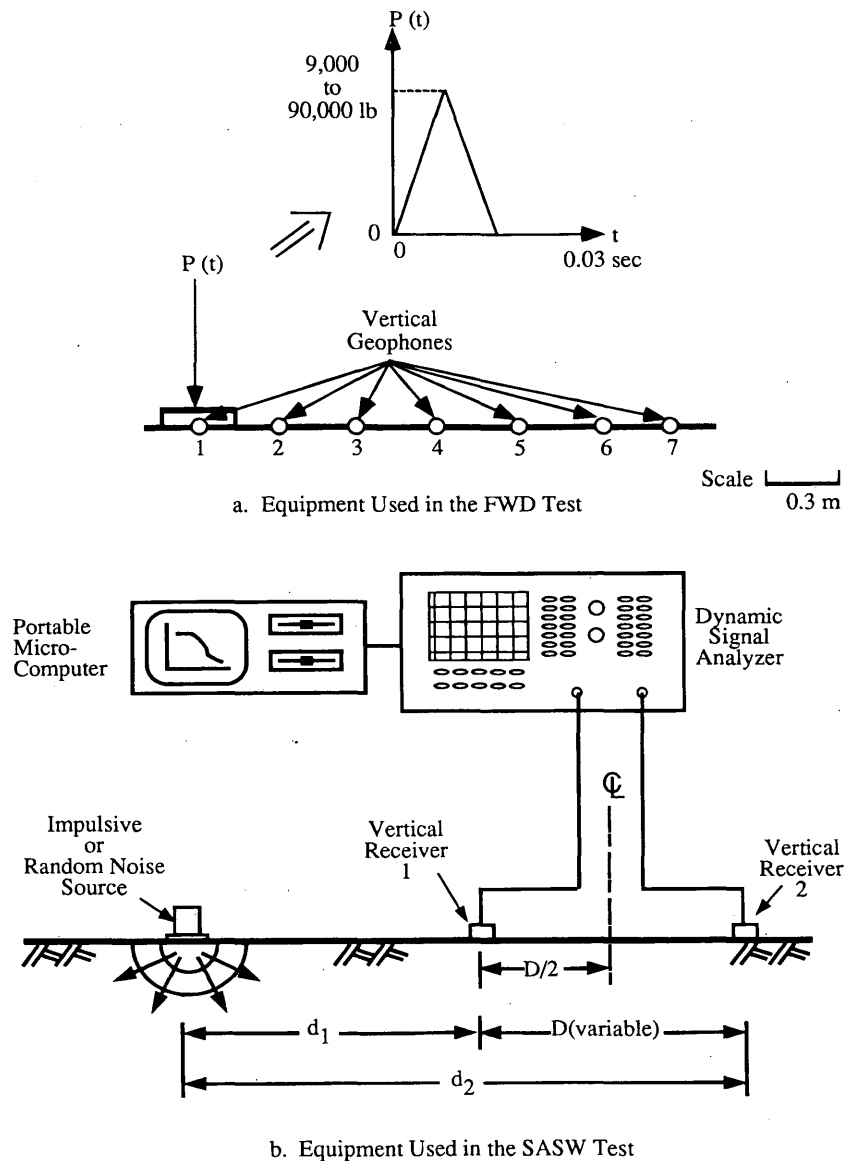


FIGURE 1 General configuration of equipment used in FWD and SASW testing of pavements.

and point load sources, and buried line and point load sources. He derived a solution to these problems through Fourier synthesis of the steady-state propagation solution. For the surface source problem, Lamb evaluated the surface displacements (horizontal and vertical), and noted that the largest disturbance in the far field is the Rayleigh surface wave. He noted the nondispersive nature of the solution, and that for a point-load excitation, the displacement decays as  $\sqrt{r}$ , where  $r$  is the distance from the source. Through the years these problems have taken on the name, Lamb's problem.

The first closed-form solution for Lamb's problem in three-dimensional space was provided by Pekeris (4) for the case a material with Poisson's ratio of 0.25. A generalization for arbitrary values of Poisson's ratio is due to Mooney (5) and also can be found in Eringen and Suhubi (6); however, the Green's functions (in the time domain) for this case are available only for a vertical point pulse with a step time-function acting on the free surface.

Miller and Pursey (7) considered the case of a circular disk vibrating harmonically and normally on the free surface of a half-space. They found explicit expressions for the displacements at points at large distances from the loaded area. These expressions for the horizontal and vertical ( $u$ ,  $w$ ) displacements at the surface of the medium due to a unit disk load are of the form

$$f(\nu) \cdot \frac{R^2}{G} \cdot \sqrt{\frac{\omega}{Cr \cdot r}} \cdot e^{-i \frac{\omega \cdot r}{Cr}}$$

where

- $R$  = the radius of the disk load,
- $G$  = the shear modulus of the medium,
- $\omega$  = the circular frequency of excitation,
- $Cr$  = the Rayleigh wave velocity of the medium, and
- $r$  = the distance to the source.

The term  $f(\nu)$  is a complex function of Poisson's ratio, for instance, for  $\nu = 1/3$ ,  $f(\nu) = -0.182(\sqrt{2}/2 + i\sqrt{2}/2)$  for the horizontal displacement, and  $0.286(\sqrt{2}/2 - i\sqrt{2}/2)$  for the vertical displacement.

While closed-form solutions to Lamb's problem have significant theoretical uses, it is improbable that exact solutions will become available soon for more complicated material or load configurations because of the mathematical difficulties involved. Thus, for the solution of dynamic problems in layered media such as pavement systems, numerical techniques must be used.

### Dynamic Loads on Layered Media: Application to the Dynamic Analysis of Pavements

Consider a pavement system that consists of horizontal layers. The mass densities and elastic moduli of the pavement system change with depth, from layer to layer, but are (assumed to be) constant over each layer. For the present application, the top layer represents the pavement surface layer (assuming it extends to infinity in both horizontal directions); the second layer is the base; and the remaining layers are the subbase layer, the soil subgrade, or both. Determining how this system responds to dynamic loads applied on the surface (or at any point within the profile) falls mathematically in the realm of wave propagation theory.

Formulation of these problems starts normally by considering steady-state harmonic forces and displacements at a given frequency. For an arbitrary transient excitation (case of the FWD test), the time history of the specified forces must be decomposed into different frequency components using a Fourier series, or more conveniently a Fourier transform. Results are then obtained for each term of the series (each frequency) and combined to obtain the time history of displacements (inverse Fourier transform).

For an isolated layer with uniform properties, the stresses and displacements along the top and bottom surfaces can be expanded in a double Fourier series (or Fourier transform) in the two horizontal directions for Cartesian coordinates, or in a Fourier series in the circumferential direction and a series of modified Bessel functions in the radial direction for cylindrical coordinates. For each term of these series corresponding to a given wave number, closed-form analytical expressions can be determined in the form of a transfer matrix relating amplitudes of stresses and displacements at the bottom surface to the corresponding quantities at the top (or vice versa). This approach [Thomson (8) and Haskell (9)] has served as the basis for most studies on wave propagation through layered media in the past 35 years. An alternative is to relate the stresses at both surfaces to the displacements, obtaining a dynamic stiffness matrix for the layer [Kausel and Roesset (10)], which can be used and understood in much the same way as in dynamic structural analysis. For a half-space, the stiffness matrix directly relates stresses and displacements at the top surface because the bottom surface is pushed to infinity.

For FWD and SASW testing of pavements with an axisymmetric load, only one term of the Fourier series is needed. The displacements  $\bar{U}$  and forces  $\bar{P}$  in the wave number domain are then related by

$$K\bar{U} = \bar{P} \quad (1)$$

where  $K$  is the dynamic stiffness matrix of the profile obtained by assembling the stiffness matrices of the layers and the underlying half-space. For a uniform vertical load applied at the surface over a

disk of radius  $R$ , the only nonzero term of the vector  $\bar{P}$  is the second term, which is equal to

$$\frac{q \cdot R}{k} \cdot J_1(kR)$$

where

- $q$  = the magnitude of the uniformly distributed load,
- $k$  = the wave number, and
- $J_1$  = the first order Bessel function.

If  $\bar{u}_1$ , and  $\bar{w}_1$  are the first two terms of the vector  $\bar{U}$ , obtained by solving Equation 1 for a vector  $\bar{P}$  with all components 0 and a 1 as the second term (for every value of  $k$ ), the surface displacements as a function of the distance  $r$  to the center of the loaded area become

$$u = q \cdot R \int_0^\infty \bar{u}_1 \cdot J_1(kR) \cdot J_1(kr) dk \quad (2a)$$

$$w = q \cdot R \int_0^\infty \bar{w}_1 \cdot J_1(kR) \cdot J_0(kr) dk \quad (2b)$$

The solution of the problem thus requires assembling the dynamic stiffness matrix  $K$  of the layered medium, solving the system of Equation 1 for many different values of  $k$  and evaluating numerically the integrals of Equation 2. The numerical integration is performed through shifting the poles of the integrand by including a small attenuation in the materials (for materials with damping, all of the poles are complex, so that no singularities are encountered along the real axis of integration). However, for systems with sharp variations in material properties between layers, the integrands may exhibit considerable waviness, making it difficult to evaluate the integrals. The solution of the equations also is time-consuming when there is a large number of layers. The procedure is convenient when dealing with a homogeneous half-space or a small number of layers.

An alternative can be obtained by expanding the terms of the dynamic stiffness matrix of a layer in terms of  $k$  and keeping terms only up to second-degree (the terms of the transfer or stiffness matrices of each layer are transcendental functions). It can be shown that this is equivalent to assuming that the displacements have a linear variation with depth over each layer using standard finite element techniques to derive the layer matrix. The stiffness matrices of each layer, the half-space, and the total profile can then be expressed in the form

$$K = Ak^2 + Bk + G - \omega^2 M \quad (3)$$

The expressions for the matrices  $A$ ,  $B$ ,  $G$ , and  $M$  are given by Kausel and Roesset (10). By computing the in-plane modes of propagation as the solution of a quadratic eigenvalue problem (11, 12) and keeping only the modes propagating outward, Kausel (13) has shown that the displacements  $\bar{u}_1$ ,  $\bar{w}_1$  in Equation 2 can be expressed as

$$\bar{u}_1 = \sum_{i=1}^{2n+2} \bar{u}_{i1} \bar{w}_{i1} \frac{k}{k_i(k^2 - k_i^2)} \quad (4a)$$

$$\bar{w}_1 = \sum_{i=1}^{2n+2} \bar{w}_{i1}^2 \frac{1}{(k^2 - k_i^2)} \quad (4b)$$

for a system with  $n$  layers, where  $\bar{u}_{i1}$  and  $\bar{w}_{i1}$  denote the horizontal and vertical displacements at the surface in the  $i$ th mode and  $k_i$  is the eigenvalue, or wave number, in the  $i$ th mode. By substituting Equation 4 in Equation 2, the integrals can be evaluated analytically in closed form (13).

This formulation requires a subdivision of the layers (thin layers are needed to accurately reproduce the variation of displacements with depth with a piece-wise linear approximation). It is particularly convenient when dealing with a large number of layers, such as when obtaining a detailed variation of soil properties with depth is desired. Furthermore, since the fundamental solutions (or Green's functions) are known explicitly, the displacements or strains at many locations can be determined without significant additional computation.

Both continuous and discrete formulations had been implemented at the University of Texas at Austin [Roesset and Shao (14), and Roesset, Stokoe, and Foinquinos (15)] to simulate the FWD and SASW tests. Although a large number of sublayers must be used in the discrete formulation to obtain satisfactory results, this formulation has been found in general to be much more efficient computationally than the continuous formulation. Therefore, the results were obtained using the discrete formulation.

## ANALYTICAL STUDIES

Two generalized pavement profiles, a flexible one and a rigid one, were selected to illustrate the dynamic response of the pavement systems to the application of FWD and SASW. Because variations in total unit weight ( $\gamma$ ), Poisson's ratio ( $\nu$ ), and damping ratio ( $D$ ) have minor effects on the dynamic response (within ranges of logical values) compared with changes in the stiffnesses of the layers, they were assumed to be the same for all the layers; that is,  $\gamma = 18,850 \text{ N/m}^3$  (120 lb/ft<sup>3</sup>),  $\nu = 0.35$ , and  $D = 0.02$ . The elastic properties and thicknesses of the layers in both profiles are given in Table 1.

### FWD Testing

#### Flexible Pavement

First the dynamic response to the FWD load of a flexible pavement with rigid rock at 6.1 m (20 ft) was computed. Figures 2(a) and 2(b) show the amplitude of the transfer function of displacements (ampli-

tude of displacements due to a unit harmonic load as a function of frequency) at Station 1, located at the center of the load, and at Station 7, which is the farthest measurement point. For low frequencies, the system behaves as if the load were applied statically. As the frequency increases, the displacements increase until they reach a peak at the same frequency at all stations. The low amplitudes of displacements at high frequencies are a result of inertial effects. These transfer functions are multiplied by the Fourier transform of the excitation and then inverted to obtain the displacement-time histories. Figure 2(c) shows the time history of displacements at each station. The main pulse is followed by oscillations with decaying amplitude, which represent the free vibrations of the complete pavement system and the soil subgrade layer in particular. These free oscillations have a well-defined period (which lies between the natural period of the subgrade for shear and compressional waves) and are essentially the same for all the recording stations. The frequency of the free vibration coincides with the frequency of the peak in the transfer functions. Chang et al. (16) have suggested a simple formula to estimate the depth to bedrock based on the free vibration period from the displacement-time records. This figure also shows that there is a time offset at the start of the motion and at the occurrence of the peak displacements at the different stations. Seng (17) has suggested the use of the offset time to find the shear velocity of the subgrade. Figure 2(d) shows the dynamic peak displacement at each station (deflection basin), obtained from the maximum displacement recorded at each receiver.

The effect of depth to bedrock on the FWD deflection basins is considered next. The analysis for the flexible pavement with different depths to bedrock is shown in Figure 3. Figures 3(a) and 3(b) show the transfer functions at Stations 1 and 7, respectively. As the depth to bedrock decreases, the peak displacement and the frequency at which it occurs increase, but the static displacement decreases. It also can be seen that the dynamic effect is more important at the farthest stations. Figures 3(c) and 3(d) show the displacement-time histories when the depth to bedrock is 6.1 m (20 ft) and when it extends to infinity. In the second case, the free oscillations are no longer present because there are no reflections from bedrock. Figure 3(e) shows that the static displacements are very sensitive to bedrock depth. However, the dynamic deflection basins are nearly independent of the depth to bedrock for depths greater than 3 m (10 ft), as shown in Figure 3(f).

#### Rigid Pavement

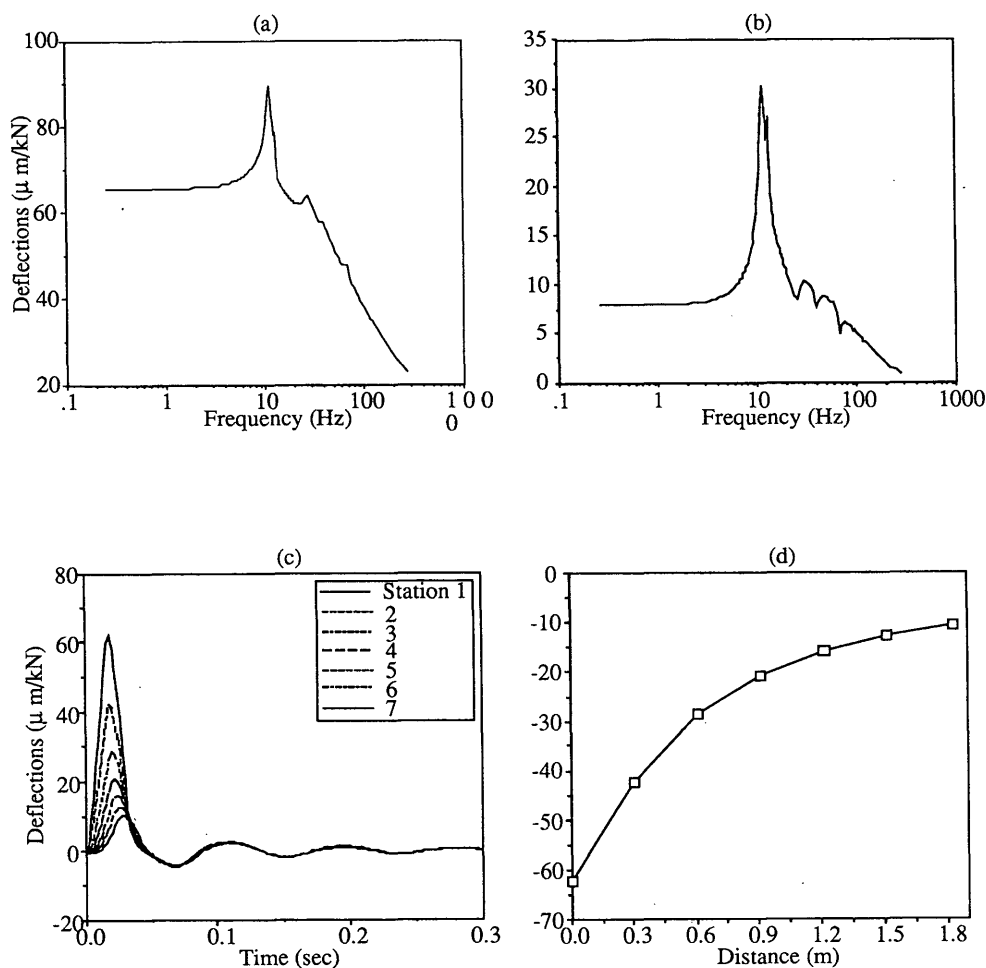
The same type of analysis was performed for the rigid pavement, and the results are shown in Figure 4. It can be observed again that the static displacements are very sensitive to the depth to bedrock, but the dynamic deflection basins are insensitive to this depth. Also, the shape and magnitude of the static and dynamic deflection basins are quite different from the ones for the flexible pavement.

#### Dynamic Amplification

The ratio of dynamic to static displacements (amplification factor) at the different stations was computed as a function of depth to bedrock. The results are shown in Figures 5(a) and 5(b) for flexible and rigid pavements, respectively. For these profiles the maximum amplification occurs at a depth to bedrock of about 2.1 to 3 m (7 to 10 ft). This means that for shallow profiles the use of a back-cal-

TABLE 1 Values of Elastic Properties and Layer Thicknesses of Generalized Pavement Profiles

Type of pavement	Layer	Thickness cm (in.)	Young's Modulus MPa (ksi)	Shear wave velocities m/sec (fps)
Flexible	Surface	15 (6)	3013 (436.7)	762 (2500)
	Base	30 (12)	483 (70)	305 (1000)
	Subgrade	variable	124 (18)	152 (500)
Rigid	Surface	22.5 (10)	39020 (5660)	2743 (9000)
	Base	15 (6)	3013 (436.7)	762 (2500)
	Subbase	30 (12)	483 (70)	305 (1000)
	Subgrade	variable	124 (18)	152 (500)



**FIGURE 2** Dynamic response of a flexible pavement with bedrock at 6.1 m to FWD loading: (a) and (b) transfer functions at recording Stations 1 and 7, (c) displacement-time histories and (d) measured deflection basin.

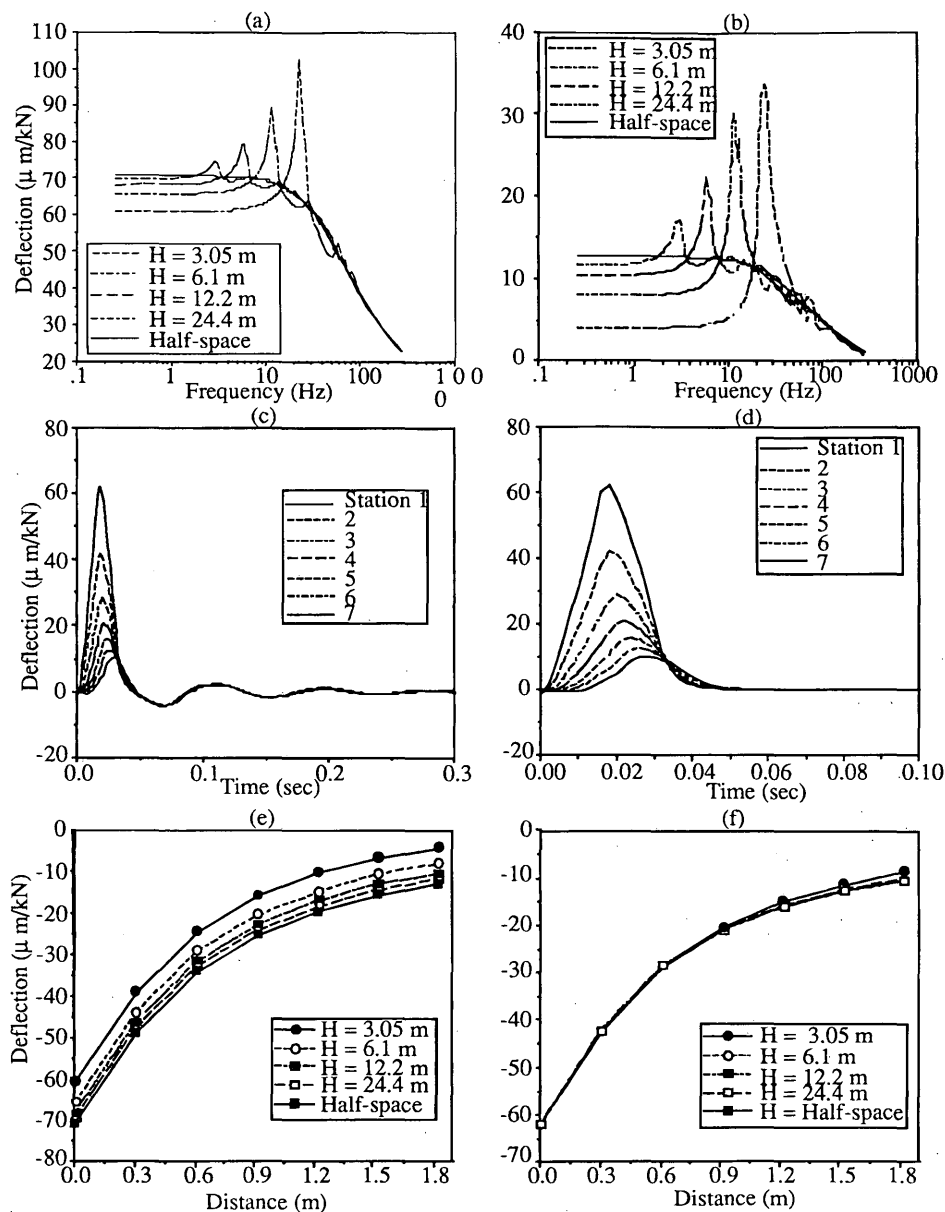
lation process based on static analysis (with a known depth to bedrock) would lead to an underestimation of the stiffness of the subgrade layer and associated complications in evaluating the other layers. For the rigid pavement, the deflection ratio becomes less than 1 for depths greater than about 4.5 m (15 ft). In this range a static back-calculation procedure would lead to an overestimation of the stiffness of the subgrade layer and associated complications in evaluating the other layers.

In static back-calculation procedures it is often assumed that the subgrade is an elastic half-space. It is therefore more interesting to compare the dynamic results for a given depth to bedrock with the static deflections for an infinite depth to bedrock. The ratios of these deflections are shown in Figures 5(c) and 5(d) for flexible and rigid pavements, respectively. The results indicate that the dynamic deflections are smaller than the static deflections for a half-space (although they can be larger than the static deflections for the same profile with a finite bedrock depth). This implies that the static back-calculation process as normally applied will lead to an overestimation of the stiffness of the layers, particularly for shallow profiles. It can also be observed that the dynamic peak displacements remain constant for a depth to bedrock greater than about 4.5 m (15 ft). This depth depends mainly on the properties of the subgrade (Seng, 1992).

### SASW Testing

The effect of depth to bedrock on the theoretical dispersion curves obtained with the SASW test are shown in Figure 6 for the flexible pavement. Figure 6(a) shows the case in which the subgrade extends to infinity. For very short wavelengths (high frequencies), the phase velocity is approximately equal to the Rayleigh wave velocity of the surface layer and remains almost constant until the wavelength approximately equals the thickness of the surface layer. For long wavelengths (low frequencies), the phase velocity approaches the Rayleigh wave velocity of the subgrade. Figure 6(b) shows the dispersion curve with a finite depth to bedrock [6.1 m (20 ft)] and Figure 6(c) shows the variation of the dispersion curves for different depths to bedrock [the rock was considered to have a shear wave velocity of 1,524 m/sec (5,000 ft/sec)]. Figure 6(c) shows that the dispersion curves start bending upward at a wavelength which is approximately equal to the depth to bedrock.

Therefore, bedrock does not affect any of the measurements performed at wavelengths shorter than the depth to bedrock. The same type of analysis was performed for the rigid pavement, and the results are qualitatively the same as those of the flexible pavement.



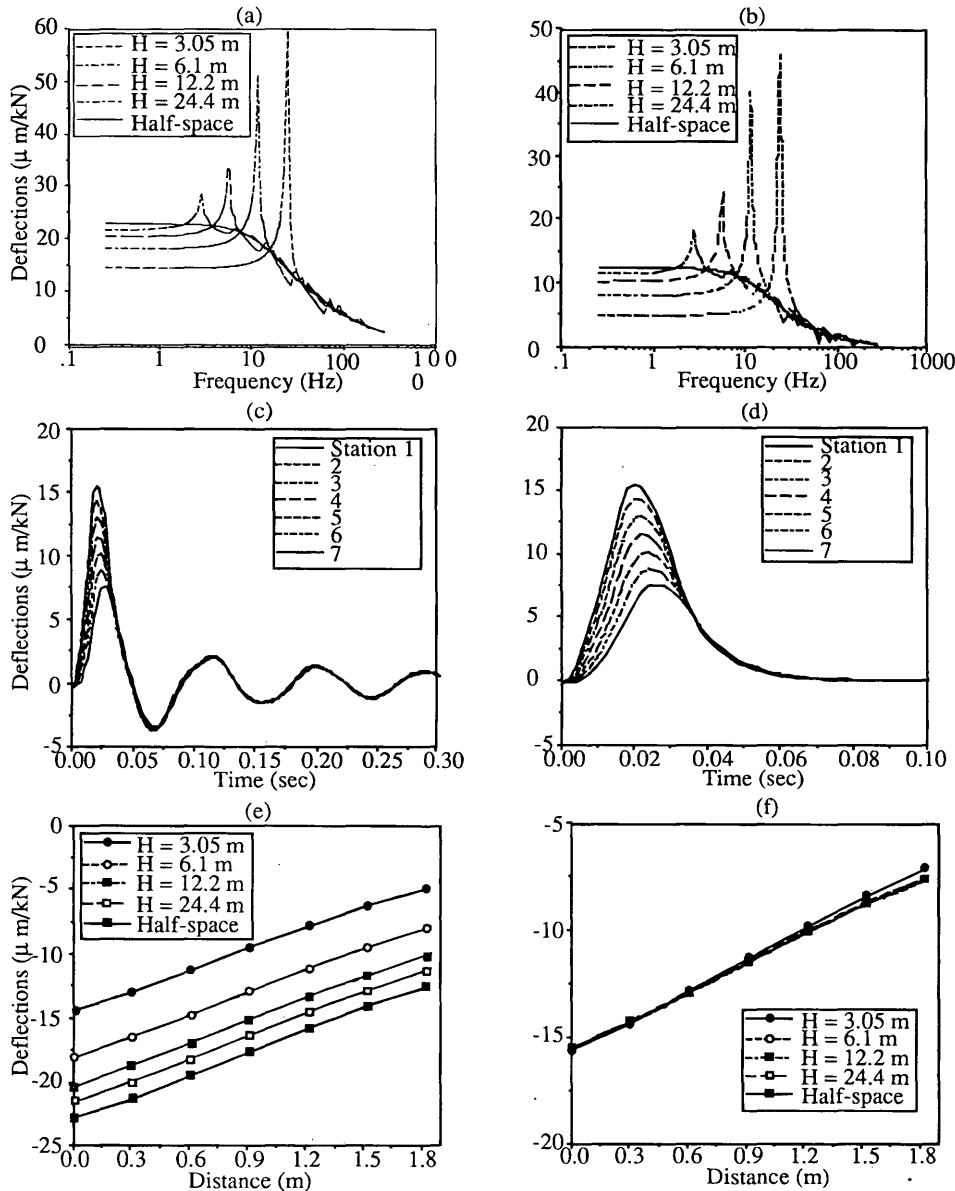
**FIGURE 3** Dynamic response of a flexible pavement with different depths to bedrock to FWD loading: (a) and (b) transfer functions at recording Stations 1 and 7, (c) and (d) displacement-time histories for depth to bedrock of 6.1 m and infinity, (e) static displacements and (f) measured deflection basins.

### Parametric Studies

The sensitivity of the deflection basins and dispersion curves to variations in the stiffnesses of the pavement layers was investigated next. The flexible pavement given in Table 1 with a subgrade extending to infinity was analyzed first. The stiffness of each layer was varied independently. Figures 7(a) and 7(b) show the deflection basins and dispersion curves for shear wave velocities of the surface layer of 381, 762, and 1,143 m/sec (1,250, 2,500, and 3,750 ft/sec), which correspond to soft, medium, and stiff pavement surface layers, respectively. Figure 7(a) indicates that the displacement under the load is influenced by the properties of the surface layer, but this

influence becomes negligible at the outer stations. Figure 7(b) shows that the dispersion curves are very sensitive to the properties of the surface layer for short wavelengths.

The deflection basins and dispersion curves when the shear wave velocities of the base are 152.4, 304.8, and 457.2 m/sec (500, 1,000, and 1,500 ft/sec) (corresponding to soft, medium, and stiff bases, respectively), are shown in Figures 7(c) and 7(d). Changes in the properties of the base affect the displacements under the load and at the next two stations, but have a negligible effect on the displacements at the last three stations. The dispersion curves are affected to some degree, but those effects are difficult to evaluate. Changes in the properties of the subgrade layer, on the other hand, affect the



**FIGURE 4** Dynamic response of a rigid pavement with different depths to bedrock to FWD loading: (a) and (b) transfer functions at recording Stations 1 and 7, (c) and (d) displacement-time histories for depth to bedrock of 6.1 m and infinity, (e) static displacement and (f) measured deflection basins.

displacements at all stations to about the same degree as shown in Figure 7(e). The dispersion curves for this case [Figure 7(f)] are very sensitive to the properties of the subgrade for long wavelengths. For the deflection basins, the displacements at the outer stations (particularly Station 7) are governed almost entirely by the properties of the subgrade layer; and for the dispersion curves, the phase velocity for short wavelengths is governed by the surface layer. The deflection basins computed with a finite depth to bedrock [6.1 m (20 ft)] were almost identical to those of Figures 7(a), (c), and (e).

The same type of parametric studies were performed for the rigid pavement. The results are qualitatively the same as those for the

flexible pavement and therefore are not shown because of space limitations.

The sensitivity of the deflection basins and dispersion curves to changes in the thicknesses of the surface and base layers was also considered. The results indicate that the deflection basins are sensitive to these changes under the load and at the near stations, but that the effect is negligible at the outer stations. The dispersion curves are sensitive to the thickness of the surface layer, but they are quite insensitive to the thickness of the base layer (within the range of logical values).

Finally, the strains were computed at various depths below the load. The results indicate that the dynamic strains are almost iden-

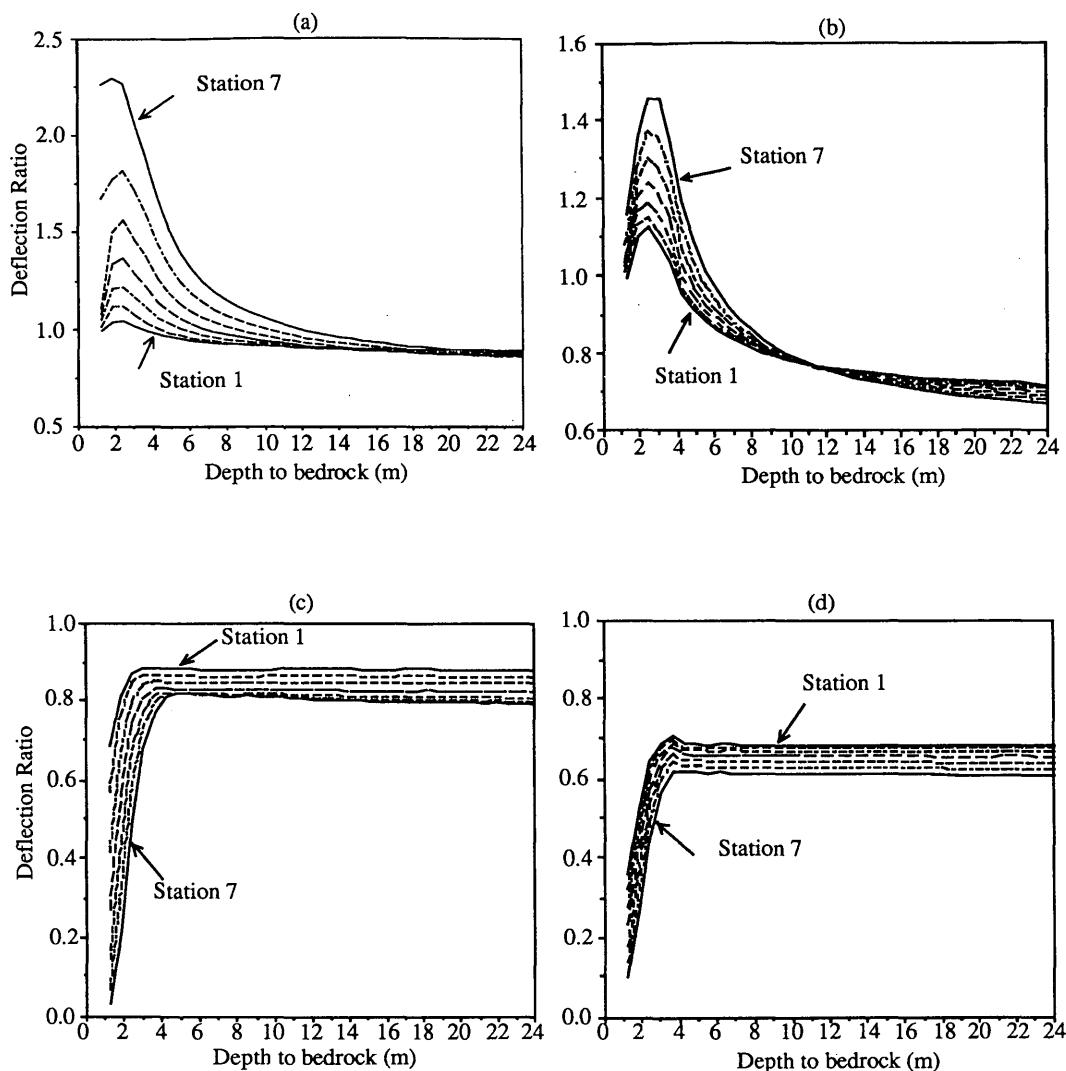


FIGURE 5 Ratio of dynamic to static displacement: static displacements with a finite depth to bedrock for (a) flexible and (b) rigid pavements; static displacements with an infinite subgrade depth for (c) flexible and (d) rigid pavements.

tical to the static ones over the top part of the pavement system and are insensitive to the depth of bedrock over this section. As the depth increases, the strains in the subgrade show more pronounced dynamic effects and a greater effect of the depth to bedrock.

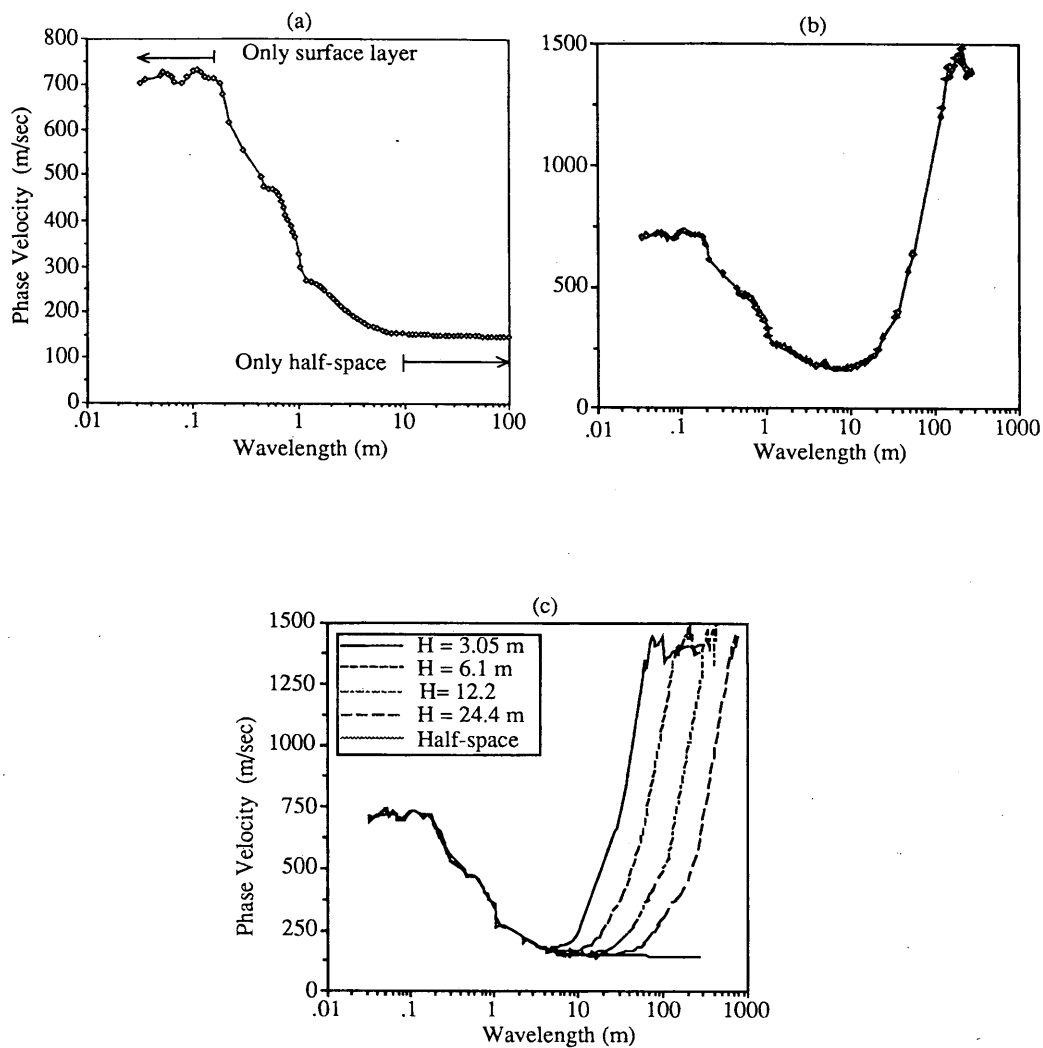
#### ADDITIONAL CONSIDERATIONS

Two of the main assumptions in the dynamic modeling of the pavement system were that the layers extend to infinity in both horizontal directions and that they are linear elastic. An accurate solution requires consideration of the finite width of the pavement and possible nonlinear behavior. These two effects were investigated in several studies conducted under the supervision of Roesset and Stokoe. Kang et al. (18) studied the effect of the finite width on the dynamic deflections of pavements and concluded that the loading position with respect to the edge of the pavement can influence (a) the amplitude of the deflections and (b) the shape of the deflection

basin obtained with the FWD test. They also showed that finite pavement width can cause some fluctuations in the dispersion curves obtained with the SASW test. However, they found that for most pavements the error committed by assuming that the pavement extends to infinity will not be serious if the load is placed more than 0.6 m (2 ft) from the edge of pavements at level sites, or 1.2 m (4 ft) from the edge when the pavement is on an embankment or a ramp with concrete retaining walls.

Chang et al. (19) studied nonlinear effects in FWD testing using an approximate nonlinear analysis procedure (a linear iterative approach in the frequency domain) and a true nonlinear incremental analysis with a generalized cap model to reproduce the nonlinear material behavior. They showed that nonlinear behavior can be significant and localized around the loaded area if testing is performed on a flexible pavement with a rather thin surface layer and a soft subgrade. However, they also showed that nonlinear effects can be neglected for small to intermediate loads for many pavement systems and that very little nonlinearity will normally be generated





**FIGURE 6** Effect of depth to bedrock on the dispersion curves measured in SASW testing of a flexible pavement.

in thick, rigid pavements. They did not study the SASW test because the small magnitude of the applied load creates only linear behavior in the pavement system.

## CONCLUSIONS

The results confirm that the dynamic response of the pavement system affects the magnitude and shape of the deflection basins obtained with the FWD test, and that these basins can be substantially different from those obtained under static conditions. When the dynamic deflection basins are compared with the static ones as a function of the assumed depth to bedrock, significant dynamic amplification is found for some range of depths to bedrock [typically less than 6.1 m (20 ft)]. However, dynamic deflections also can be smaller than the static ones over a wide range of depths to bedrock [typically depths greater than 15 m (50 ft)]. The use of a static back-calculation procedure with the known bedrock depth will result in an underestimation of the stiffness of the subgrade layer in the first case and an overestimation in the second case. The

stiffnesses evaluated for the other layers will be complicated by the errors in the subgrade layer. On the other hand, current static back-calculation practice is to consider that the subgrade extends to infinity (because the depth to bedrock is not normally known). In this case, the static back-calculation procedure will result in a general overestimation of the stiffness of the pavement system.

The current method for interpreting FWD test results fails to utilize the test's true potential because the method does not consider the dynamic nature of the pavement response. Recording a longer time history of the dynamic response of the pavement system would yield a simpler and faster estimation of the subgrade stiffness from the offset times. The depth to bedrock also could be estimated from the period of the pavement's free vibrations, which follow the passage of the FWD pulse. This information and the history of the force are needed if a dynamic back-calculation procedure is to be used for system identification.

Computation of strains under the axis of the FWD load reveals that the static and dynamic strains are almost the same in the upper layers and that the effect of depth to bedrock is negligible for the strains in the top part [the upper 0.6 m (2 ft)] of the pavement. The dynamic

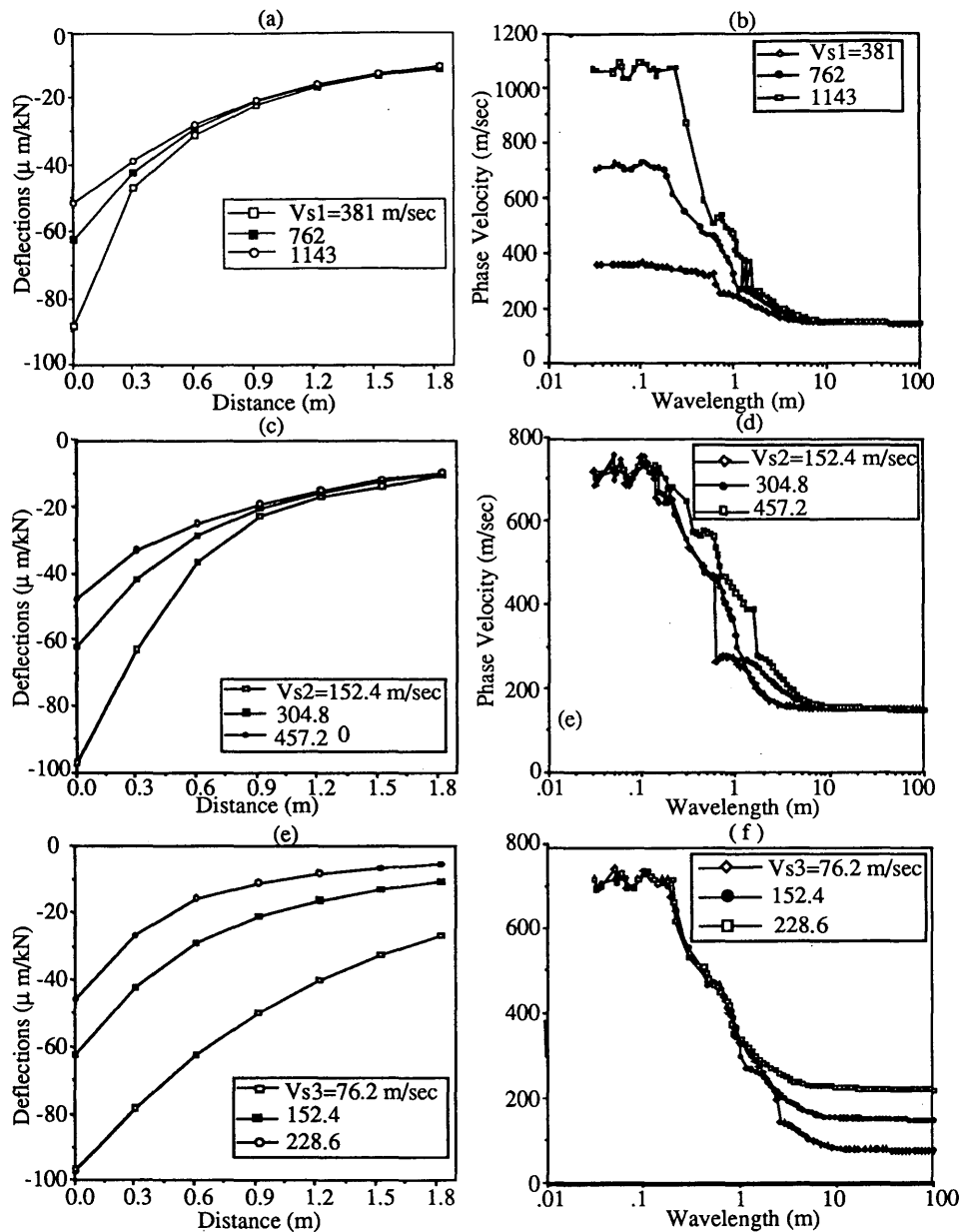


FIGURE 7 Sensitivity of deflection basins and dispersion curves to changes in the stiffness of the layers: (a) and (b) surface layer, (c) and (d) base layer, (e) and (f) subgrade layer.

effect on the strains increases with depth. The parametric studies clearly indicate that the dynamic deflections at the outer receivers are governed almost exclusively by the subgrade properties.

The SASW test, on the other hand, is particularly efficient in determining the properties of the top layer (stiffness and thickness). For long wavelengths, the phase velocity approaches the Rayleigh wave velocity of the subgrade. If there is a rocklike material at some depth, the dispersion curve starts bending upward at a wavelength of the order of the depth of the bedrock, and its presence is clearly seen in the dispersion curve. Unfortunately, SASW dispersion curves are relatively insensitive to the properties of the base material.

Using both the FWD and SASW tests is advised, since the research shows that the two tests complement each other.

#### ACKNOWLEDGMENTS

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