Effect of Depth to Bedrock on Deflection Basins Obtained with Dynaflect and Falling Weight Deflectometer Tests

Der-Wen Chang, Yumin Vincent Kang, Jose M. Roesset, and Kenneth H. Stokoe II

The dynaflect and falling weight deflectometer (FWD) are commonly used for nondestructive testing of pavements. In both cases a dynamic load is imparted on the surface of the pavement, and deflections are measured at various points along the surface. Evaluation of the moduli of the surface layer, base, and subgrade is normally performed by comparing the experimental deflections with the results of static analyses. The moduli of the layers in the static model are then varied in an iterative procedure until a reasonable match between experimental and theoretical deflections is obtained. This solution ignores the dynamic nature of these nondestructive tests. In this paper the effect of depth to bedrock on the amplitude of the deflections and the shape of the deflection basins obtained with the dynaflect and the FWD tests is investigated analytically. Dynamic and static deflections at four different pavement profiles are compared. The results show that the range of bedrock depths over which dynamic effects are important differs between the two nondestructive tests because of the excitation frequencies and depends mainly on the stiffness of the subgrade. The results also show that, when dynamic effects occur in the measurements but are not taken into account in the analysis, the modulus of the subgrade is generally underestimated, sometimes by 50 percent or more, and the moduli of the base and surface layer are overestimated. Finally, a simple method is suggested for the FWD that makes it possible to estimate the depth to bedrock by recording the free vibration of the pavement system.

Reliable measurements of the in situ conditions of pavements are an important aspect in effectively managing pavement systems. The dynaflect and falling weight deflectometer (FWD) are two devices commonly used in practice for nondestructive determination of the values of Young's moduli for the various layers in a pavement profile.

The dynaflect consists of a force generator and five geophones housed in a small trailer that is towed by a light vehicle. The loading system consists of two counter-rotating eccentric masses. The resulting vertical force varies harmonically with time. At a frequency of 8 Hz, a 1,000-lb peak-to-peak oscillating force is transmitted to the pavement through the loading wheels. The resulting deflection basin is measured by five geophones mounted on the trailer draw bar at 12-in. intervals.

The position of the geophones (Stations 1 through 5) with respect to the wheels is shown schematically in Figure 1.
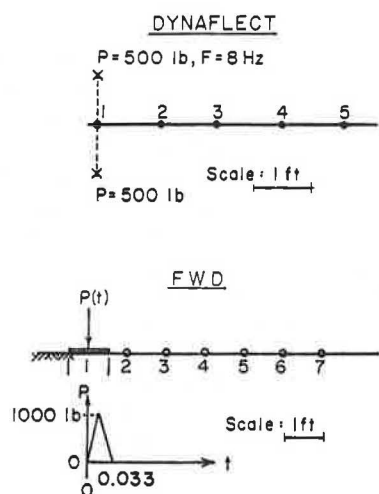
The FWD has a drop weight mounted on a vertical shaft and housed in a compact trailer that can easily be towed by most conventional vehicles. The drop weight is hydraulically lifted to predetermined heights ranging from about 2 to 20 in. The weight is then dropped onto a spring system that helps condition and distribute the load to the loading area. The resulting load is a force impulse with a duration of approximately 30 msec and a peak magnitude ranging from about 2,000 lbs to more than 20,000 lbs, depending on the drop height, drop weight, and pavement stiffness. The peak force and maximum deflections at various points along the surface are measured by a load cell and a set of velocity transducers. The applied pressure is measured in kilopascals and the deflections in micrometers. The positions of the load and recording stations are also shown in Figure 1, the dynaflect in plan and the FWD in profile views.

In the case of the dynaflect the deflections measured at the various stations represent the amplitudes of the steady-state displacements at a given frequency (8 Hz). For the FWD they are the peak displacements under a transient type excitation. In both cases the tests are dynamic in nature, but interpretation of the results to estimate the moduli of the surface layer, base, and subgrade relies on static analyses. Furthermore, these analyses assume that the soil in the subgrade is an elastic, uniform half-space or an elastic stratum of finite thickness. In many cases soil properties vary with depth, and the soil is underlain at some depth by much stiffer, rock-like material that causes reflections in the stress waves imparted during the dynamic tests.

The purpose of this work is to assess the effect of depth to bedrock on the amplitude of the deflections and the shape of the deflection basins that would be measured by the dynaflect and FWD for four different pavement profiles. The influence of the dynamic effects on backcalculated moduli is presented. Finally, the possibility of recognizing the existence of bedrock at a finite depth and estimating this depth from the measured data is discussed.

FORMULATION

Static and dynamic analyses of pavement systems always assume the existence of horizontal layers with different material

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**FIGURE 1 Geometric configuration
of loads and stations for dynaflect
and FWD simulations.**

properties that extend to infinity in the horizontal directions.
The top layer represents the surface layer, the second represents the base, and the following layers represent the soil of
the subgrade. An accurate solution requires consideration of
the finite width of the pavement, but for the purpose of this
work the above assumptions are not unreasonable. The effects
of the finite width of the pavement and the position of the
load with respect to the edge were the subject of a previous
paper (*1*). Determination of the response of this system to
dynamic loads applied at the surface falls mathematically into
the area of wave propagation theory.

The formulation of these problems always starts by considering steady-state harmonic forces and displacements at a
given frequency. For a harmonic excitation as applied by the
dynaflect the solution at the desired frequency (typically 8
Hz) directly provides the desired results. For an arbitrary
transient excitation as applied by the FWD the time history
of the load must be first 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 are later combined to obtain
the time history of displacements (inverse Fourier transform).
When the frequency considered is zero, the results of the
analysis are the static solution.

For a given frequency the solution proceeds by considering
a uniform single layer with two horizontal surfaces, at the top
and bottom. Due to a vertical load, displacements, stresses,
and strains at any point will be independent of the angle q
(circumferential direction) in cylindrical coordinates, and a
function only of the radial distance from the vertical axis
through the origin. They can then be expressed in a series of
Bessel or modified Bessel functions. Each term of the series
represents a wave number. For each wave number one can
then develop a dynamic stiffness matrix for the layer relating
stresses at the two interfaces to the displacements at the same
elevations. By assembling the stiffness matrixes of the different layers (surface layer, base, and subgrade), one can form
a total stiffness matrix for the pavement system and compute
the displacements at any point due to the applied load (for
each wave number). The actual displacements are then ob-
tained by performing an inverse Bessel (or Hankel) transform,
involving an integral from zero to infinity of the displacements
for each wave number multiplied by a Bessel function. This
provides the steady-state solution for each frequency. In FWD
tests the process is repeated for many different frequencies.
The time history of the displacements is then obtained by
applying an inverse Fourier transform. For the dynaflect the
last step is not needed, and results are obtained only for one
frequency (typically 8 Hz).

The terms of the dynamic stiffness matrixes of each layer
involve transcendental functions (*2*). An alternative is to divide the physical layer into a number of thin sublayers. For
these sublayers one can approximate the variation of the displacements with depth by a straight line (or higher order
polynomial expansions if so desired). This approximation leads
to much simpler algebraic expressions for the terms of the
stiffness matrixes, which allows one to compute the wave
numbers (eigenvalues) and mode shapes (eigenvectors) of the
waves propagating through the pavement system by solving
an algebraic eigenvalue problem (*3,4*). The displacements
caused by harmonic dynamic loads can then be obtained in
explicit form (without the need to compute the Bessel integrals) as shown by Kausel (*5*).

Each one of these two approaches (continuous and discrete
formulations) has advantages and disadvantages from a computational point of view. Both have been implemented in
computer programs that yield nearly identical results. In the
discrete solution the division of the physical layers into sublayers is done automatically inside the program on the basis
of a number of parametric studies conducted by Roesset and
Shao (*6*), so that the input is the same for all the programs.

The results presented here were obtained using the computer programs UTDYNAF and UTFWD developed at the
University of Texas at Austin to simulate the dynaflect and
the FWD, respectively, using the discrete formulation.

PARAMETRIC STUDY

Roesset and Shao (*6*) conducted a number of parametric studies on a hypothetical pavement profile, comparing the dynamic deflections with those that would be obtained from
static analyses assuming that the subgrade extended to infinity
(an elastic half space) and considering a rigid base at a finite
depth (the same depth used for the dynamic analyses). Davies
and Mamlouk (*7*) and Mamlouk (*8*) used the same formulation to conduct parametric studies and investigate the importance of dynamic effects and reached similar conclusions.
Davies and Mamlouk (*7*) suggested in particular a formula
for the frequency at which the maximum dynamic amplification takes place as a function of the thickness and shear
wave velocity of the subgrade. In this study four actual sites
were considered to conduct further analyses.

Description of Test Sites

Four typical in-service pavement sections were selected as the
models for the test sites in these analyses. The models are
patterned after (but do not exactly match): FM 137 in Paris,
Texas; FM 195 in Paris, Texas; Route 1 in Austin, Texas;

and Interstate highway 10 in El Paso, Texas. Detailed information used for the calculations on these test sites is presented in Table 1. The damping ratios were assumed to be 2 percent in all layers. Variation of the damping ratios from these assumed values will obviously alter the proportions of the results of this study. Each profile is modeled as a horizontally layered stratum resting over bedrock (rigid rock). The depth to bedrock, defined as the total depth measured from the surface to the bedrock, was varied by changing the thickness (h) of the subgrade.

Effect of Depth to Bedrock on Deflection Basins for Dynaflect Tests

Simulation of the dynaflect test was conducted on the four selected profiles, and the corresponding static solutions were obtained for comparison purposes. Figure 2a shows the ratio of dynamic to static deflections as a function of the assumed depth to bedrock for the first profile (FM 137). The dynamic deflection is the amplitude of the steady state response, which is also the peak deflection in this range. It can be seen that, for all stations, the deflection ratio initially starts from a value of nearly one at shallow depths, reaches its maximum at a critical depth of about 27.5 ft, and finally decreases to one as the depth increases. There is a second smaller peak at a depth of about 70 ft. The dynamic amplification factor is not constant at all receiver stations. The maximum deflection ratio occurs at the fifth station (farthest from the source) and reaches a maximum value of 2.23 at the critical depth (27.5 ft). The depths at which resonance would occur for pure shear and compression waves due to a harmonic excitation at a frequency of 8 Hz are indicated in Figure 2a by symbols d_s and

d_p, respectively. It can be seen that the critical depth is located between d_s and d_p, but is quite close to the depth corresponding to resonance associated with compression waves, d_p.

Figures 2b, 2c, and 2d show the deflection ratios for the FM 195, Route 1, and I-10 profiles. The results for the second and fourth sites (FM 195 and I-10) are similar to those of Site 1, with important dynamic amplification occurring when the depth to bedrock is from 15 to 35 ft. On the other hand, for the third profile (Route 1), large amplifications occur when the depth to bedrock is between 30 and 65 ft. It is interesting to notice that the only difference between the first two sites is the thickness of the surface layer (1 in. versus 4 in.). The thickness of the base and the elastic moduli of the surface layer, base, and subgrade are otherwise identical. The fourth site has different moduli and thicknesses for the pavement and base, in addition to having an additional subbase layer. However, the properties of the subgrade are the same as Profiles 1 and 2. On the other hand, the subgrade at Profile 3 has a shear wave velocity that is twice that of the other sites (Young's modulus is about 4 times higher than that of the other sites). Comparison of the four sites indicates that the dynamic effects are primarily influenced by the stiffness and thickness of the subgrade.

Dynamic effects have been traditionally represented in structural dynamics by amplification factors that represent the ratio of peak dynamic to static displacements (or forces). This is the approach followed in Figure 2. The advantage of this approach is that it shows clearly at what frequency, or for what depth to bedrock in the case of a fixed frequency, dynamic effects are important. For the present application it is interesting, however, to look also at the difference between dynamic and static deflections, instead of their ratio. This is shown in Figure 3 for the four sites. The differential deflections are measured in mils (thousandths of inches). It can be observed that, in general, the deflection differences are fairly uniform over the 5 stations for a given depth to bedrock. (The exceptions are at 30 ft for Profiles 1, 2, and 4, and 60 ft for Profile 3, where the difference increases with distance from the source.) From a physical point of view these results imply that the dynamic effects can be interpreted as a train of plane compressional waves traveling down the profile, reflecting at the rigid bottom and then at the (free) pavement surface. From a practical point of view this implies that the deflection basin obtained experimentally could be corrected for dynamic effects by subtracting a constant quantity from the deflections measured at all five stations. Unfortunately the amount to be subtracted is a function of the thickness and modulus of the subgrade and of the effective damping. Therefore, it is difficult to estimate this additional increment without previous knowledge of the properties of the pavement system. Moreover, application of a correction to obtain a static basin is of limited interest because dynamic analysis of a layered pavement system at a fixed frequency is no more expensive nor time-consuming than the usual static analysis.

The main limitation of the dynaflect is that the test is conducted at a single frequency and that only the amplitudes of the steady-state vibrations are used. If one were to conduct the test at two different frequencies (e.g., 8 and 16 Hz), it would at least allow a comparison of the measured deflection basins. If the results were almost identical for the two frequencies, one could conclude that dynamic effects are not

TABLE 1 Properties of Four Test Sites Used as Models

| Site	Layer	Thickness (in.)	S-Wave Velocity (fps)	P-Wave Velocity (fps)	Unit Weight (pcf)	Damping Ratio (%)
FM 137	AC	1	2500	3900	140	2
Paris, Texas	base	12	1000	1730	125	2
	subgrade	h*	500	1000	110	2
FM 195	AC	4	2500	3900	140	2
Paris, Texas	base	12	1000	1730	125	2
	subgrade	h	500	1000	110	2
Route 1	AC	7	3000	5200	145	2
Austin, Texas	base	6	1000	2080	130	2
	subgrade	h	1000	2080	130	2
Interstate	CRC	10	9000	14030	145	2
Highway 10,	base	6	2500	3900	145	2
El Paso,	subbase	12	800	1600	125	2
Texas	subgrade	h	500	1000	125	2

* h is a variable
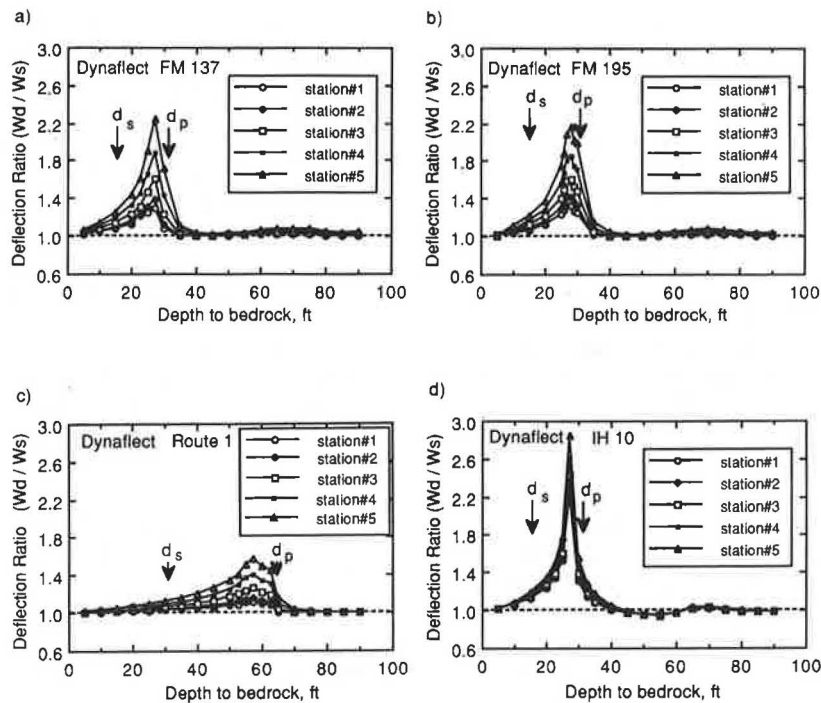
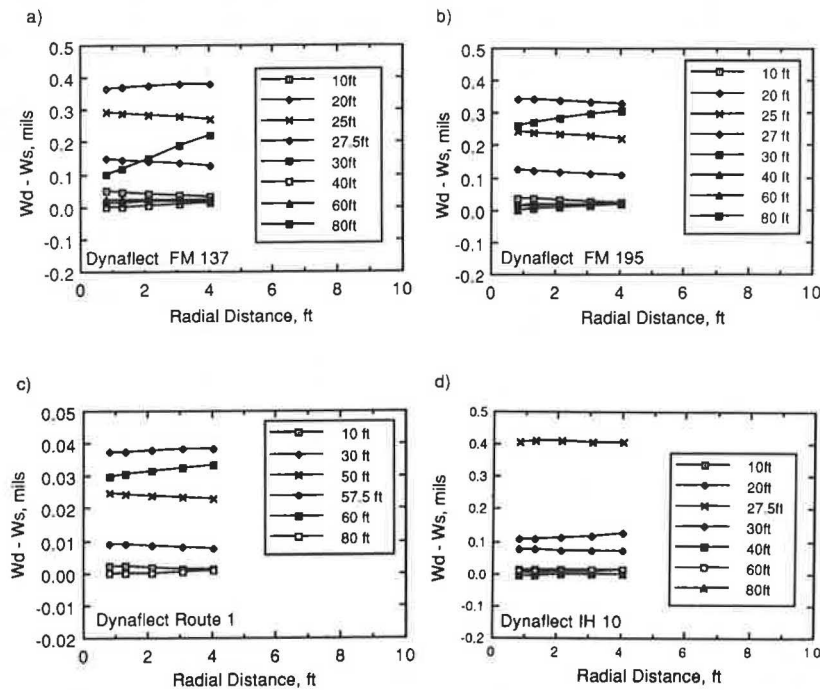**FIGURE 2 Deflection ratios versus depth to bedrock for dynaflect testing.**

FIGURE 3 Difference in deflections versus radial distance for dynaflect testing.

important, that bedrock is located at a sufficient depth (of the order of 70 ft or more), and that a static analysis would be appropriate to backcalculate the elastic properties of the layers. On the other hand, if the results showed clear differences, it would be apparent that dynamic effects are important. In this case testing at other frequencies would be needed

in order to define the peak of the amplification function and to permit estimation of the depth to bedrock. A simpler alternative might be to continue recording the motions at the different stations after the excitation has ceased in order to obtain the free vibrations of the pavement system. In this case a procedure similar to the one proposed for the FWD in the

following section could be used to estimate, relatively simply, the depth to bedrock.

Effect of Depth to Bedrock on Deflection Basins for FWD Tests

The results of the simulations of the FWD tests are presented again in terms of deflection ratio (dynamic amplification factor) as a function of the assumed depth to bedrock in Figure 4. The dynamic deflection is the peak deflection. Two main points are notable. First, the critical depth to bedrock for the FWD (depth at which the maximum amplification occurs) is always less than for the dynaflect. This result is because the FWD generates higher frequencies than the dynaflect. If one defines a characteristic or predominant frequency as the natural frequency of the pavement system in compression for the depth to bedrock at which the maximum dynamic amplification takes place, the predominant frequency of the FWD test is between 30 and 35 Hz (versus 8 Hz for the dynaflect). Second, the deflection ratio can become less than one for the FWD, especially for the deeper subgrade, whereas for the dynaflect, the ratio is almost always greater than one. It should be noted that when the deflection ratio is less than one, the use of an inversion process based on a static analysis will lead to overestimating the Young's moduli of the layers as discussed in the next section.

For Profile 1 (Figure 4a) the maximum amplification occurs for a depth to bedrock of about 5 ft (between 5 and 7.5 ft). The critical depth is about 7.5 ft for Profile 2 (Figure 4b) and Profile 4 (Figure 4d) and becomes of the order of 15 ft for Profile 3 (Figure 4c). This trend is consistent with that observed for the dynaflect and reflects the higher modulus of

the subgrade in Route 1 (third profile). At the third site the amplification is higher than one over the complete range of depths to bedrock studied. At Profile 1 the amplification is slightly less than one at all stations for depths to bedrock of 60 ft or more and is smaller than one over most of the range of depths under the load (Station 1). The results are similar for Profile 2, although the deamplification is slightly more pronounced. For Profile 4 the dynamic deflections are smaller than the static ones at all stations for depths to bedrock greater than 20 ft. It is interesting to notice that in this range the ratio of dynamic to static deflection is essentially the same at all stations (the deflection basin is multiplied by a constant factor).

Figure 5a shows the differences between dynamic and static deflections for the FWD and Profile 1. It can be seen that in this case the differences are not constant for all stations as in the case of the dynaflect. This implies that the dynamic effects are more complex because of the many frequencies involved and can no longer be explained on the basis of plane waves. Similar results are obtained at the other sites (Figures 5b, 5c, and 5d).

Effects of Bedrock Depth on Moduli Backcalculated from FWD Data

Knowledge of the effects of bedrock at a finite depth on the FWD deflection measurements can be used to investigate the resulting effects on backcalculated layer moduli. This was done by presuming that the peak dynamic deflections could be equated to the static FWD deflections. The peak dynamic deflections were then used with conventional interpretation procedures to backcalculate layer moduli. For simplicity the following study was conducted by choosing two cases for each
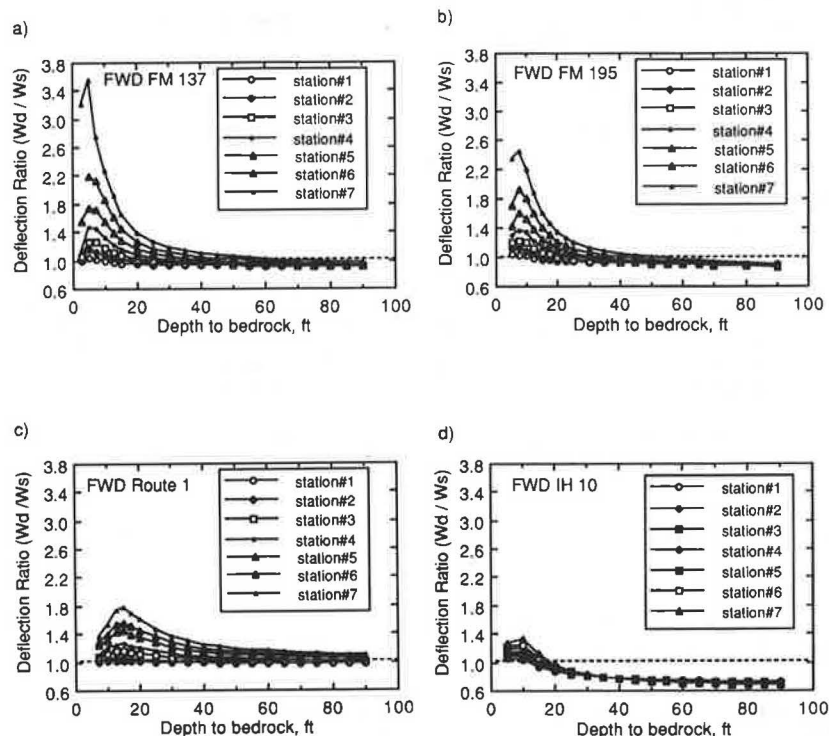

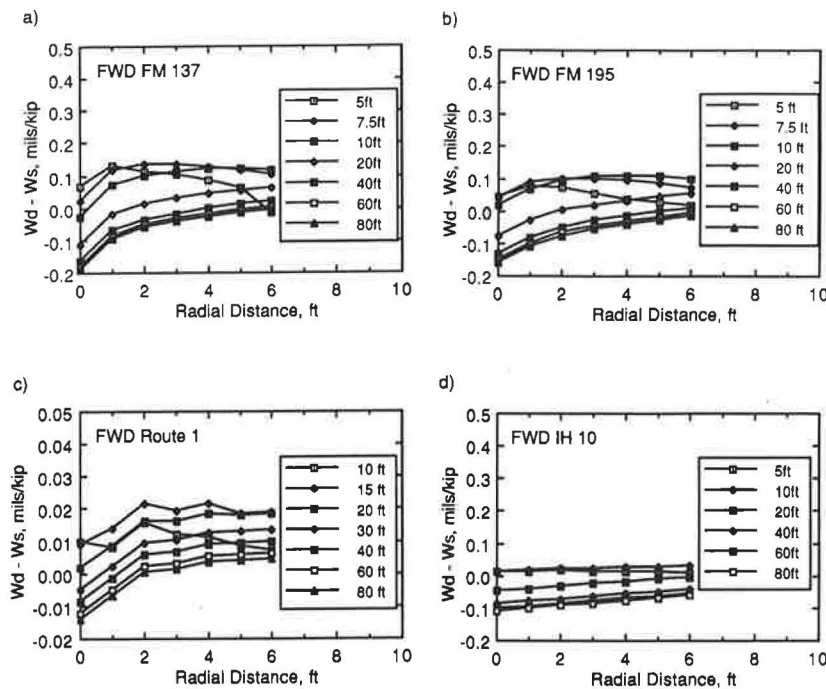**FIGURE 4 Deflection ratios versus depth to bedrock for FWD testing.**

FIGURE 5   Difference in deflections versus radial distance for FWD testing.

site (see Table 2) in which the associated depth to rock results in either (a) peak dynamic amplification or (b) an insignificant amount of dynamic amplification or possibly even some de-amplification (such as the bedrock at 80 ft). By assigning the dynamic FWD deflection basin to the basin-fitting program MODULUS (9), layer moduli were backcalculated and compared with their original values. The resulting errors in terms of the ratio of backcalculated moduli to their original values are summarized in Table 3.

For the FM 137 site deflections at peak amplification result in a 50 percent underestimation of the subgrade modulus,

TABLE 2   Estimated Depth to Bedrock for Four Study Sites with Various Bedrock Depths

Site	Measured Period (sec) (T_n)	Estimated Depth to Bedrock (ft)	Actual Depth to Bedrock (ft)
1	0.02	6	5
	0.03	9	7.5
	0.04	12	10
	0.08	24	20
2	0.02	6	5
	0.03	9	7.5
	0.04	12	10
	0.06	18	20
3	0.02	6	10
	0.03	9	15
	0.04	12	20
	0.07	21	30
4	0.02	6	5
	0.04	12	10
	0.09	27	20

NOTE: Estimated depth to bedrock based on $h = 300\ T_n$. Compression wave velocity in the subgrade is assumed to be 1200 fps. Accuracy of the estimated depth increases in direct proportion with the accuracy of the assumed compression wave velocity of the subgrade.

whereas the moduli of the surface and base layer are over-estimated by about 100 percent. When the depth to bedrock is 80 ft at the same site the modulus of the surface layer is greatly overestimated whereas the moduli of the base and subgrade are only overestimated by 6 percent, an insignificant amount. For the FM 195 site the peak amplification results in a 37 percent underestimation of the subgrade modulus, about a 60 percent overestimation for both the surface layer and base moduli. When the depth to the bedrock is 80 ft, overestimations of the moduli of the surface, base, and subgrade layers are all less than 20 percent. For Route 1 use of peak displacements gives 3 percent and 21 percent underestimations for the surface and subgrade layers. However it gives a 107 percent overestimation for the base moduli. In the case in which the dynamic amplification ratio is nearly one dynamic effects are found to give -1 percent, $+33$ percent, and -3 percent errors in layer moduli for Route 1. It is interesting to note that even in this case, the static analyses give results in error in the modulus of the base layer. For the I-10 site the peak amplification results in 25 percent and 135 percent overestimations of the surface and subbase moduli and 4 percent and 33 percent underestimations for the moduli of the base and subgrade. In the case in which the depth to the bedrock is deep as 80 ft the significant dynamic deampli-fication phenomenon results in 8 percent, 31 percent, 115 percent, and 44 percent overestimations for the moduli of the surface, base, subbase, and subgrade layers, respectively. All these results are based on the assumed 2 percent damping.

Generally speaking, conventional (static) interpretation of the FWD data appears to yield overpredictions of the material stiffness for the surface and base layers. This is particularly true in the cases in which the dynamic effects on the measured response are important (shallow depth to the rock). In this case the stiffness of the subgrade is significantly underpre-

TABLE 3 Comparison of Backcalculated Layer Moduli Using Computer Program Modulus with Actual Moduli Used to Generate Dynamic Deflections

| Site	Depth (ft)	AC Surface				Base				Subgrade			
		True E_o	Input E_x	Comp. E_c	E_c/E_o	True E_o	Input E_x	Comp. E_c	E_c/E_o	True E_o	Input E_x	Comp. E_c	E_c/E_o
FM 195	5	434	200 to 1000	803	1.85	57	25 to 200	145	2.16	16	16	8.0	0.5
	80	434	200 to 1000	5082	11.71	67	25 to 200	59	0.88	16	16	17	1.06
FM 137	7.5	434	200 to 1000	700	1.61	67	25 to 200	111	1.66	16	16	10	0.63
	80	434	200 to 1000	443	1.02	67	25 to 200	80	1.19	16	16	18	1.13
Route 1	15	704	200 to 1000	680	0.97	76	25 to 200	157	2.07	76	76	60	0.79
	80	704	200 to 1000	699	0.99	76	25 to 200	101	1.33	76	76	74	0.97

Site	Depth (ft)	CRC Surface				AC Base				Subbase				Subgrade			
		True	Input	Comp.	E_c/E_o	True	Input	Comp.	E_c/E_o	True	Input	Comp.	E_c/E_o	True	Input	Comp.	E_c/E_o
IH 10	10		1000 to 10000	7263	1.25	450	200 to 1000	431	0.96	46	25 to 200	108	2.35	18	18	12	0.67
	80	5826	1000 to 10000	6286	1.08	450	200 to 1000	591	1.31	46	25 to 200	99	2.15	18	18	26	1.44

dicted. If the dynamic response of the FWD test is significantly less than the presumable static response, such as the cases of the I-10 site at which depths to the rock are greater than 20 ft, the stiffness of the subgrade can be overpredicted significantly.

To overcome the uncertainties involved in using the static FWD interpretation procedure knowledge of the depth to bedrock and materials damping is important as is the dynamic nature of the test. A simple way of assessing bedrock depth is described in the next section.

Estimation of Bedrock Depth from FWD Tests

Although in the normal use of the FWD only the peak deflections are kept to construct the deflection basin and then backfigure the elastic moduli, one can obtain at each station the complete history of the motions as a function of time. Figure 6a shows the deflection time histories at the first station (under the load) for the fourth site (I-10) assuming a deep subgrade (simulating a half space) and bedrock at a depth of 10 ft. In the case of the half-space (dotted line) there is basically a displacement pulse similar in shape to the applied load. In the case of bedrock at a depth of 10 ft, on the other hand, the main pulse is followed by several oscillations, with decaying amplitude, which represent the free vibrations of the pavement system and the subgrade layer in particular. These residual vibrations have a well-defined period that corresponds approximately to the natural period of the profile in compression. Moreover, as shown in Figure 6b, this period

a)

b)
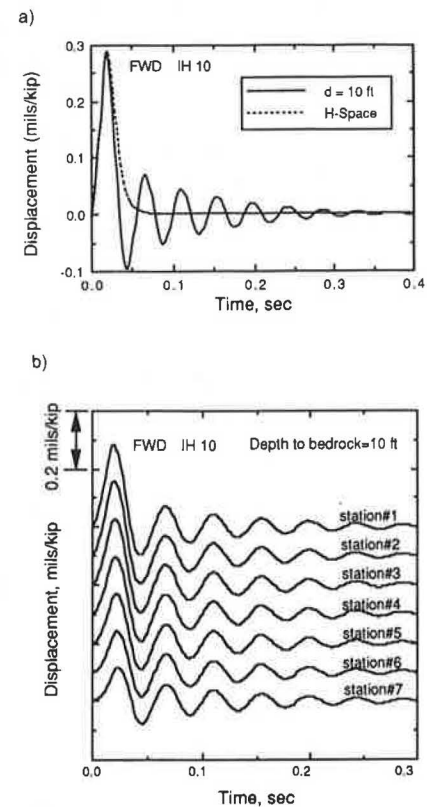
FIGURE 6 Time histories of FWD displacements for Profile 4 (I-10).

is essentially the same at all recording stations. For the I-10 site with a depth to bedrock of 10 ft the period is about 0.04 sec. Similar results were obtained for different depths to bedrock (with the corresponding change in the period) and for the other sites. This suggests that the depth to bedrock can be easily estimated from a simple observation of the displacement-time records. The natural frequency of interest is approximately

$$f_n = 1/T_n \cong c_p/4h \tag{1}$$

where c_p is the compressional wave velocity of the subgrade and h is the depth to bedrock. As a first approximation and before any inversion is performed one can assume a shear wave velocity for typical subgrades on the order of 600 ft/sec. (For the sites considered here the shear wave velocity of the subgrade varied from 500 ft/sec for Sites 1, 2, and 4 to 1,000 ft/sec for Site 3.) By assuming an average Poisson's ratio of 0.33, the compressional wave velocity would be $c_p = 1,200$ ft/sec. Then

$$h \cong c_p/4T_n \cong 300 \ T_n \tag{2}$$

with the period in seconds and the depth to bedrock h in feet, or

$$h \cong 90 \ T_n \tag{3}$$

with h in meters.

Davies and Mamlouk's (7) formula would yield $h = 0.2 \ c_p \ T_n$, which differs only by 20 percent.

By applying this formula to the free vibrations shown in Figure 6, one would obtain a depth to bedrock of 12 ft instead of the actual 10 ft used for the analyses. Table 2 presents the measured periods from time records, estimated depths, and actual depths for various other cases. Because the shear wave velocity of the subgrade is 500 ft/sec for Profiles 1, 2, and 4, the formula should overestimate the depth to bedrock by about 20 percent. For Profile 3 the shear wave velocity was 1,000 ft/sec, and the formula should underestimate the depth to bedrock by about 40 percent. It is important to notice that this preliminary estimate can be obtained directly in the field without any need for computer processing of the data. It can then be used in the inversion process to obtain improved estimates of the moduli. Once Young's modulus of elasticity of the subgrade has been determined, an improved estimate of the depth to bedrock can be obtained as

$$h \cong 60 \ \sqrt{E} \ T_n \tag{4}$$

with E in ksi, T_n in seconds, and h in ft, or

$$h \cong \tfrac{1}{4} \sqrt{E} \ T_n \tag{5}$$

with E in kN/m^2 and h in meters.

These formulas assume an average unit weight of 120 lb/ft^3 (20000 N/m^3) for the subgrade and an average Poisson's ratio of 0.33. They are, therefore, approximations intended primarily to obtain an order of magnitude estimate. The value of the Poisson's ratio assumed for all the formulas is appropriate for average subgrades that are not 100 percent saturated.

Additional studies are being conducted to select more appropriate coefficients when the subgrade is fully saturated. However, the general approach is simple and should be helpful in evaluating bedrock depth for all subgrade materials.

CONCLUSIONS

The results of the studies presented in this paper show clearly that dynamic effects can influence the magnitude of the deflections and the shape of the deflection basin obtained in the dynaflect and FWD tests. The importance of these effects is a function of the depth to bedrock and the material properties (including material damping) when there is a sharp discontinuity in the values of the elastic modulus of the subgrade and that of the underlying rock. For the dynaflect test these effects lead to significant dynamic amplification when bedrock is at a depth ranging from about 20 to 60 ft for small values of damping, depending on the properties of the subgrade. Determination of the moduli of the surface layer, base and subgrade using static analyses, as is the current practice, may then lead to a significant underestimation of the modulus of the subgrade and an overestimation of the moduli of the base and surface layer. For the FWD dynamic amplifications occur only for much smaller depths to bedrock, on the order of one-fourth of the dynaflect's (5 to 15 ft typically). On the other hand, there can be deamplifications (dynamic deflections smaller than the static ones) over a wide range of depths for the FWD. In this case static analyses would generally overestimate the elastic moduli of all layers in the pavement.

It appears that using only the maximum deflections recorded at each station fails to use the true potential of the FWD. Recording the complete time history of the deflections, at least at one station, allows a simple and fast estimation of the depth to bedrock, which can be performed in the field simply by measuring the period of the residual vibrations after the first pulse. From the decay of these residual vibrations, one could also estimate an effective damping. This same concept could be extended to the dynaflect.

ACKNOWLEDGMENTS

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