

Determination of Depth to Bedrock from Falling Weight Deflectometer Test Data

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Depth to bedrock can have a significant effect on the moduli of the surface layer, base, and subgrade back-calculated from Falling Weight Deflectometer (FWD) tests when the calculations are performed using a static analysis. On the other hand, when the back calculation is based on dynamic analysis of the FWD test, the resulting moduli are quite insensitive to the depth to bedrock if this depth is larger than about 3 m (10 ft). When only the peak deflections at the seven receivers are recorded, there is little that can be done analytically to account for the depth to bedrock. However, when the time histories of the motions at the various receivers are stored, one can estimate both the depth to bedrock and the modulus of the subgrade rather easily from the time histories. These estimations are most correctly applied to time histories of 0.12 sec or longer rather than the 0.06 sec conventionally recorded. In this article, the results of a number of parametric studies simulating the FWD test on three flexible pavements and one rigid pavement are presented. From these results, approximate expressions were developed for (a) estimating the depth to bedrock, (b) computing the natural period of vibration of the pavement system, which is nearly equal to that of the subgrade layer, and (c) estimating the wave propagation velocity and modulus.

Deflection basins caused by the dynamic loads imposed by the Falling Weight Deflectometer (FWD) are nearly independent of depth to bedrock if this depth is larger than about 3 m (10 ft). Deflection basins caused by the same load applied statically are, on the other hand, affected by the existence of bedrock to depths of 15 m (50 ft) or more. As a result, if the back-calculation procedure used to determine the elastic moduli of the surface layer, base, and subgrade employs a static analysis, the results will depend on the depth to bedrock as shown by several studies over the last decade. The objective of this article is to discuss procedures by which the depth to bedrock and the modulus of the subgrade can be estimated directly from the FWD results, without back calculation, when the time history of the motions recorded at the various receivers is provided rather than only the peak deflections.

Analytical simulations of the FWD test were conducted using the UTFWD program (1,2). The four pavement profiles shown in Figure 1 were studied. The main purposes of these studies were as follows:

1. To determine the depth to bedrock at which resonance may occur for various stiffnesses of the subgrade, including unsaturated and saturated conditions;
2. To develop equations for estimating this resonant depth to bedrock;
3. To develop a method for estimating the depth to bedrock based on the free vibrations of the pavement system created in the FWD test; and

4. To develop an approach for estimating the stiffness of the subgrade layer based on the phase shift between the first pulses in the deflection-time records of the FWD test at two measurement stations.

The deflection basins presented in this article are normalized with respect to the load. Therefore, actual deflections under any load are simply calculated by multiplying the normalized deflection by the desired load.

MODEL PARAMETERS

Only the subgrade stiffness and thickness were varied in this study. The stiffnesses of the other pavement layers were kept constant as listed.

1. Continuously reinforced concrete (CRC):
 - shear wave velocity, $V_s = 2,593$ m/sec (8,500 fps)
 - Young's modulus, $E = 37.4$ MN/m² (5,425 ksi)
2. Asphalt concrete (AC):
 - $V_s = 915$ m/sec (3,000 fps)
 - $E = 4.8$ MN/m² (690 ksi)
3. Base material:
 - $V_s = 305$ m/sec (1,000 pfs)
 - $E = 0.46$ MN/m² (67 ksi)

The thickness of each of these upper pavement layers varied between the four profiles but it was kept constant in each profile.

The shear wave velocity of the subgrade layer in Profiles 1 to 4 was varied from 150 to 450 m/sec (500–1,500 fps), and the corresponding Young's modulus varied from 0.11 to 0.98 MN/m² (16 to 142 ksi). In addition, the depths to bedrock of the four pavement profiles (Figure 1) were varied from 1.65 to 27 m (5.5–90 ft), which simply means that the thickness of the subgrade was varied. The stiffness of the bedrock beneath the subgrade was assumed to be infinite.

To simulate an unsaturated subgrade, a Poisson's ratio of 0.33 was used. The material properties of the four pavement profiles with unsaturated subgrade conditions are given in Table 1. To simulate a saturated subgrade, the compression (P-wave) velocity of the subgrade was set equal to 1,525 m/sec (5,000 fps). This velocity represents typical field conditions for uncemented saturated soils (3). The shear wave (S-wave) velocities of the saturated subgrades were varied from 150 to 450 m/sec (500–1,500 fps), as in the unsaturated subgrade condition. As a result, Poisson's ratio varied from 0.495 to 0.451 as the S-wave velocity of the subgrade varied from 150 to 450 m/sec (500–1,500 fps). Hence, Young's modulus (which is equal to the resilient modulus) for the saturated subgrade varied from 0.12 to 1.07 MN/m² (18–155 ksi). The material properties of the four pavement profiles with saturated subgrades are given in Table 2.

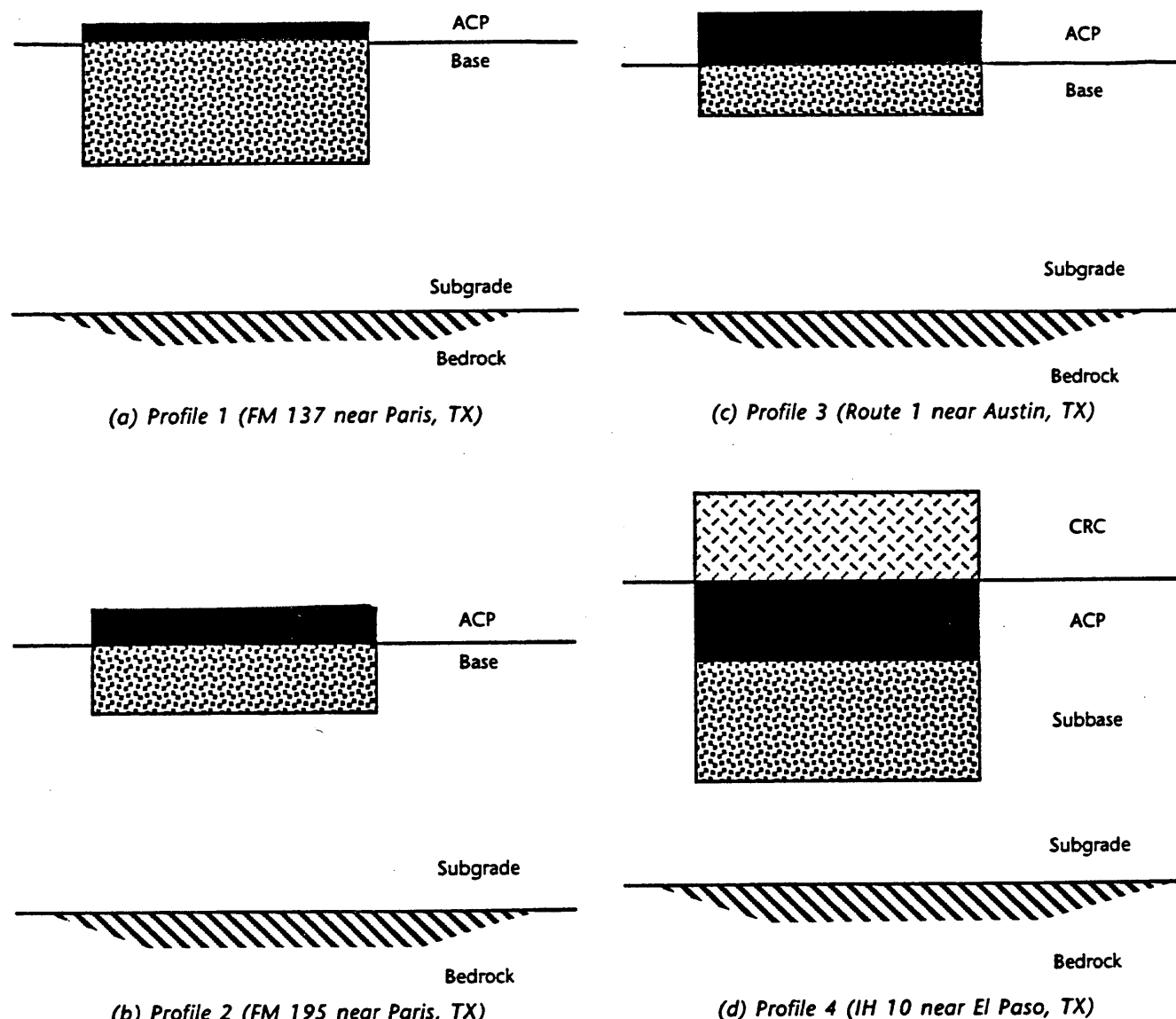


FIGURE 1 Idealized cross-section of the four pavement profiles.

DEFLECTION BASINS

A typical example of the deflection basins created by the FWD test as a function of depth to bedrock is shown in Figure 2. These results correspond to Profile 1 with a soft, unsaturated subgrade. The dynamic deflections at all seven measurement stations (receivers) used in conventional testing vary only at shallow bedrock depths of less than 3 m (10 ft) as shown in Figure 2(a). An important observation is that the deflection at the first measurement station (under the load) remains nearly constant throughout the entire range of bedrock depths. Only at depths less than about 2 m (7 ft) is this deflection affected. On the other hand, the dynamic deflections at shallow bedrock depths are influenced more (relative to Station 1) as the distance from the load point increases.

The static deflections at all seven measurement stations for Profile 1 with an unsaturated subgrade are shown in Figure 2(b). As can

be seen comparing Figure 2(a) and 2(b), static deflections are influenced more by shallow bedrock than dynamic deflections. In addition, they are influenced over a wider range of bedrock depths than dynamic deflections. These two observations are critical points when considering the dynamics in the FWD test.

An easy way to view the effect of bedrock depth on the FWD test is in terms of deflection ratio; that is, the ratio of dynamic deflections to static deflections. The deflection ratios for Profile 1 with an unsaturated subgrade are shown in Figure 2(c). In this case, there is a peak in the deflection ratios at a depth of about 1.8 m (6 ft). The depth to bedrock corresponding to the maximum deflection ratio is called the resonant depth to bedrock (4). It is at this depth where back calculation using a procedure that assumes static loading in the FWD test would be most in error.

A typical set of plots showing the combined effects of subgrade stiffness and depth to bedrock on the deflection ratios measured at

TABLE 1 Simplified Equations for Estimating Depth to Bedrock with Unsaturated Subgrade Conditions

a) FM137 (Profile 1)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	1	0.27	140	0.02	3,000	5,345	690.4
Base	12	0.25	125	0.02	1,000	1,732	67.4
Subgrade*	h**	0.33	110	0.02	500	993	15.8
		0.33	110	0.02	750	1,488	35.5
		0.33	110	0.02	1,000	1,985	63.1
		0.33	110	0.02	1,500	2,978	142.0
b) FM195 (Profile 2)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	4	0.27	140	0.02	3,000	5,345	690.4
Base	6	0.25	125	0.02	1,000	1,732	67.4
Subgrade	h	0.33	110	0.02	500	993	15.8
		0.33	110	0.02	750	1,488	35.5
		0.33	110	0.02	1,000	1,985	63.1
		0.33	110	0.02	1,500	2,978	142.0
c) Route 1 (Profile 3)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	7	0.27	145	0.02	3,000	5,345	715.1
Base	6	0.25	130	0.02	1,000	1,732	70.1
Subgrade	h	0.33	130	0.02	500	993	18.7
		0.33	130	0.02	750	1,489	42.0
		0.33	130	0.02	1,000	1,985	74.6
		0.33	130	0.02	1,500	2,979	167.9
d) IH 10 (Profile 4)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
CRC	10	0.2	145	0.02	8,500	13,880	5424.2
ACP	6	0.27	145	0.02	3,000	5,345	715.1
Base	12	0.25	125	0.02	1,000	1,732	67.4
Subgrade	h	0.33	110	0.02	500	993	15.8
		0.33	110	0.02	750	1,488	35.5
		0.33	110	0.02	1,000	1,985	63.1
		0.33	110	0.02	1,500	2,978	142.0

* There are four different S-wave velocities for each subgrade.

** Thickness of subgrade (h) was varied from 5.5 to 90 ft.

all seven stations in the FWD test is presented in Figure 3. These results are for Profile 1 with an unsaturated subgrade. As seen in the figure, the resonant depth to bedrock increases as the stiffness of the subgrade increases. Also, it should be noted that Station 7 in the FWD test is most affected by shallow bedrock and Station 1 (at the center of the load) is least affected.

Dynamic and static deflection basins as a function of distance from the source obtained at the peak deflection ratio (the resonant depth to bedrock) are shown in Figure 4 for Profile 1 (flexible pavement) with unsaturated and saturated subgrade conditions, respectively. As can be seen, there is little difference at the source between

dynamic deflections and static deflections for Profile 1 with the softest subgrade [Figure 4(a)]. The differences between dynamic deflections and static deflections become larger as the distance from the source increases. This behavior explains why the deflection ratio at the nearest station is the smallest and the deflection ratio at the farthest station is the largest, as illustrated in Figures 2(c) and 3.

For the stiffest subgrade condition, there is little difference between the dynamic deflections and the static deflections, as shown in Figure 4(b). For the saturated subgrade conditions at Profile 1, the trends are similar to those first described for the unsaturated subgrade conditions, as shown in Figure 4(c) and 4(d).

TABLE 2 Simplified Equations for Estimating Depth to Bedrock with Saturated Subgrade Conditions

a) FM 137 (Profile 1)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	1	0.270	140	0.02	3,000	5,345	690.4
Base	12	0.250	125	0.02	1,000	1,732	67.4
Subgrade*	h**	0.495	110	0.05	500	5,000	17.4
		0.489	110	0.05	750	5,000	39.5
		0.479	110	0.05	1,000	5,000	69.8
		0.451	110	0.05	1,500	5,000	154.8
b) FM 195 (Profile 2)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	4	0.270	140	0.02	3,000	5,345	690.4
Base	6	0.250	125	0.02	1,000	1,732	67.4
Subgrade	h	0.495	110	0.05	500	5,000	17.7
		0.489	110	0.05	750	5,000	39.5
		0.479	110	0.05	1,000	5,000	69.8
		0.451	110	0.05	1,500	5,000	154.8
c) Route 1 (Profile 3)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
ACP	7	0.270	145	0.02	3,000	5,345	715.1
Base	6	0.250	130	0.02	1,000	1,732	70.1
Subgrade	h	0.495	130	0.05	500	5,000	21.0
		0.489	130	0.05	750	5,000	46.7
		0.479	130	0.05	1,000	5,000	82.5
		0.451	130	0.05	1,500	5,000	183.0
d) IH 10 (Profile 4)							
Material Type	Thickness (in.)	Poisson's Ratio	Unit Weight (pcf)	Damping Ratio	S-wave Velocity (fps)	P-wave Velocity (fps)	Young's Modulus (ksi)
CRC	10	0.200	145	0.02	8,500	13,880	5424.2
ACP	6	0.270	145	0.02	3,000	5,345	715.1
Base	12	0.250	125	0.02	1,000	1,732	67.4
Subgrade	h	0.495	110	0.05	500	5,000	17.7
		0.489	110	0.05	750	5,000	39.5
		0.479	110	0.05	1,000	5,000	69.8
		0.451	110	0.05	1,500	5,000	154.8

* There are four different S-wave velocities for each subgrade.

**Thickness of subgrade (h) was varied from 5.5 to 90 ft.

Similar plots of deflection basins as a function of distance from the source for Profile 4 (rigid pavement) are shown in Figure 5 for unsaturated and saturated subgrade conditions. The amplitudes of the deflection basins are much smaller than those obtained with Profile 1, because the surface layer of Profile 4 is much thicker and stiffer than that of Profile 1. There is also less variation in the deflection basins with distance from the FWD load than in Profile 1. This response can be most easily seen by comparing Figures 4(a) and 5(a). This difference occurs because Profile 4 represents a rigid pavement, whereas Profile 1 represents a very flexible pavement. For saturated subgrade conditions [(Figure 5(c))], the trends for the

rigid pavement are similar to those just described for the unsaturated subgrade conditions [(Figure 4(c))].

RESONANT DEPTH TO BEDROCK

The analytical simulations of the FWD tests for the four pavement profiles were expressed in terms of deflection ratios as a function of depth to bedrock. The resonant depth to bedrock, RD_b , for each pavement profile with each subgrade stiffness was determined as the depth to bedrock corresponding to the maximum deflection

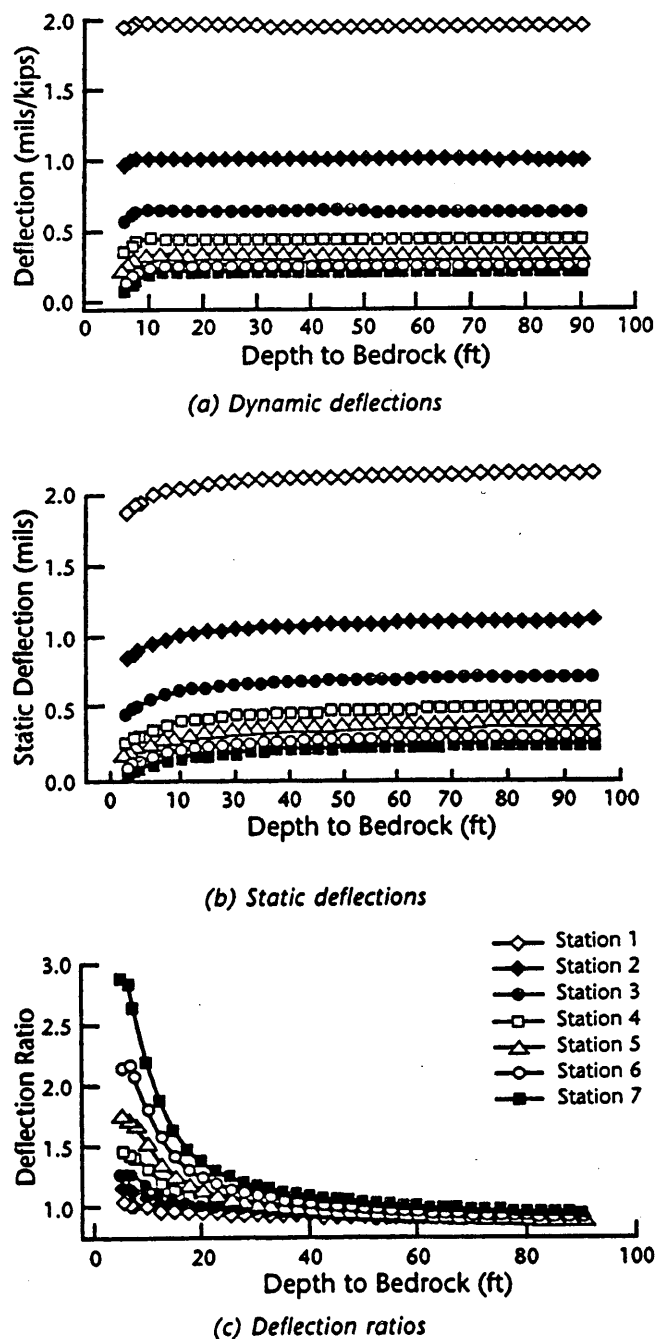


FIGURE 2 Deflection basins obtained with FWD testing for Profile 1 with unsaturated subgrade conditions [V_s of subgrade = 500 fps (155 m/sec) and $E = 16$ ksi (0.11 MN/m²)].

ratio. The values of RD_b as a function of various subgrade stiffnesses for the four pavement profiles are summarized in Figure 6. The resonant depths to bedrock obtained with the FWD test varied from 1.7 to 6.2 m (5.5–20 ft). The values of RD_b were plotted versus subgrade stiffness, and the results are presented in Figure 7.

As seen in Figure 7, the resonant depths to bedrock form two groups; one for flexible pavements (Profiles 1, 2, and 3) and the

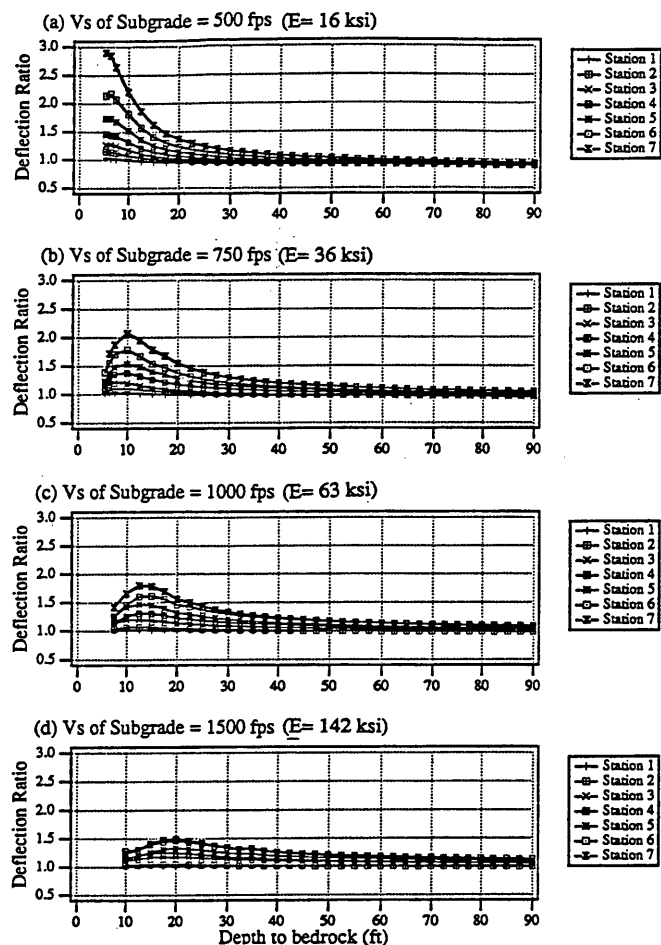


FIGURE 3 Deflection ratio versus depth to bedrock for FWD tests at Profile 1 with unsaturated subgrade conditions.

other for the rigid pavement (Profile 4). For flexible pavements, a straight line was fitted to the data, which results in:

$$RD_b = 0.013V_s \quad (1)$$

For the rigid pavement, the fitted line was:

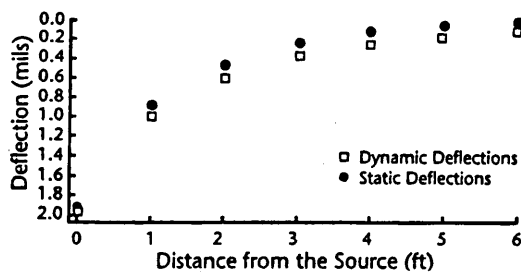
$$RD_b = 0.011V_s \quad (2)$$

In Equations 1 and 2, RD_b and V_s can be expressed in any consistent set of units. It is easy to see that the resonant depth to bedrock can be predicted from the subgrade stiffness. Equation 1 can also be expressed in terms of Young's modulus (E) as:

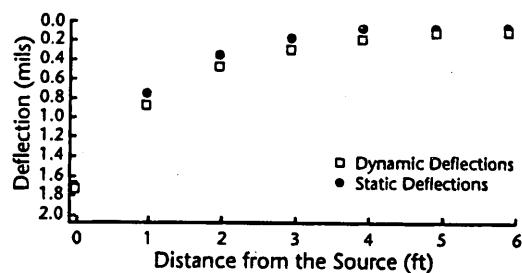
$$RD_b = 0.00018 \sqrt{E} \quad (0.0043 \sqrt{E}) \quad (3)$$

for flexible pavements with the unit weight of the subgrade assumed to be 19 800 N/m³ (110 lb/ft³) and Poisson's ratio of the subgrade assumed to be 0.33. Likewise, Equation 2 can be expressed as:

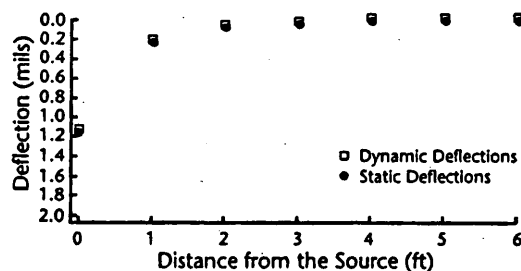
$$RD_b = 0.00015 \sqrt{E} \quad (0.0036 \sqrt{E}) \quad (4)$$



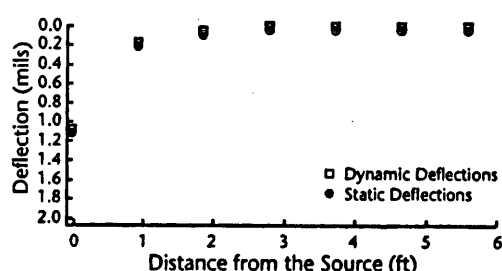
(a) Unsaturated subgrade with S-wave velocity = 500 fps ($E = 16$ ksi)



(c) Saturated subgrade with S-wave velocity = 500 fps ($E = 16$ ksi)

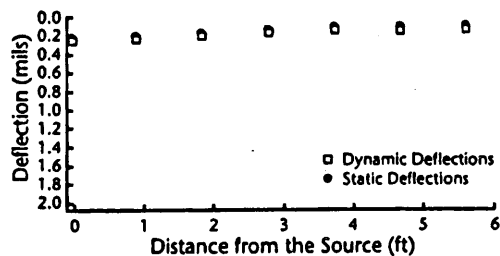


(b) Unsaturated subgrade with S-wave velocity = 1,500 fps ($E = 142$ ksi)

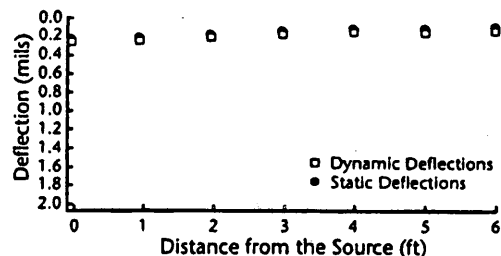


(d) Saturated subgrade with S-wave velocity = 1,500 fps ($E = 142$ ksi)

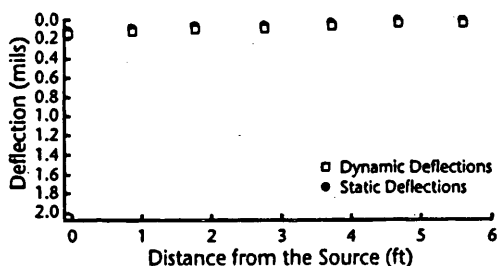
FIGURE 4 Deflection basins as a function of distance from the FWD source for Profile 1 with saturated subgrade conditions.



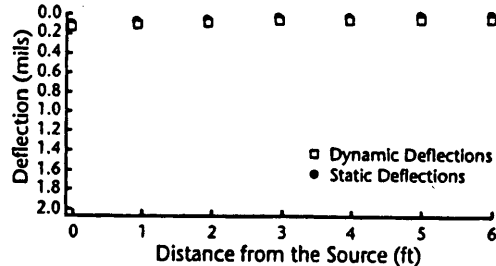
(a) Unsaturated subgrade with S-wave velocity = 500 fps ($E = 16$ ksi)



(c) Saturated subgrade with S-wave velocity = 500 fps ($E = 16$ ksi)



(b) Unsaturated subgrade with S-wave velocity = 1,500 fps ($E = 142$ ksi)



(d) Saturated subgrade with S-wave velocity = 1,500 fps ($E = 142$ ksi)

FIGURE 5 Deflection basins as a function of distance from the FWD source for Profile 4 with unsaturated subgrade conditions.

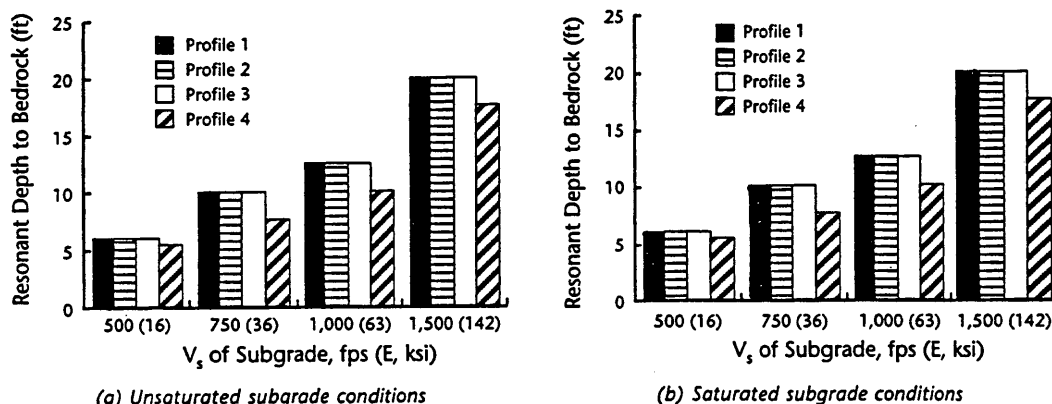


FIGURE 6 Resonant depths to bedrock for FWD testing at the four pavement profiles with various subgrade stiffnesses.

for the rigid pavement. In Equations 3 and 4, E is in N/m^2 (or psf) and RD_b in m (or ft).

AMPLITUDE OF MAXIMUM DEFLECTION RATIO

The amplitude of the deflection ratio at each measurement station is an important index of the potential error generated in any static interpretation procedure (4). Figure 8 shows the maximum deflection ratios as a function of various subgrade stiffnesses for the four pavement profiles. It should be noted that the maximum deflection ratio of the FWD test always occurs at the farthest measurement station (Station 7).

As can be seen in Figure 8, the amplitudes of the maximum deflection ratios of the four pavement profiles decrease as the stiffness of the subgrade increases for both unsaturated and saturated subgrade conditions. This means that the accuracy of back calculated layer moduli based on a static interpretation method should improve as the subgrade stiffness increases (4).

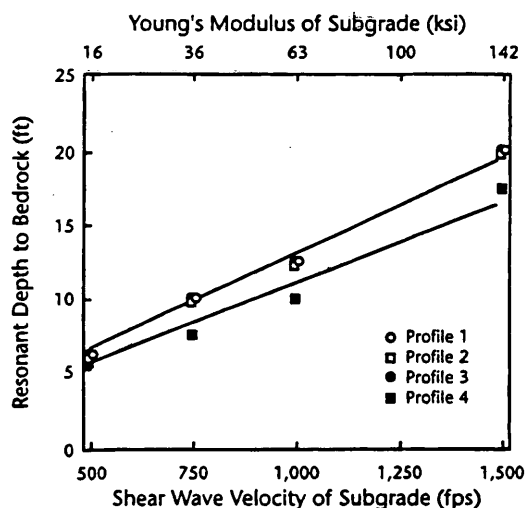


FIGURE 7 Curve fitting of the resonant depth to bedrock for FWD testing at the four pavement profiles with various subgrade stiffnesses.

The estimated amplitudes of the maximum deflection ratios of the four pavement profiles obtained with saturated subgrade conditions are generally larger than those obtained with unsaturated subgrade conditions. This indicates that the back-calculated layer moduli obtained at pavement sites with unsaturated subgrade conditions based on a static interpretation method should be more

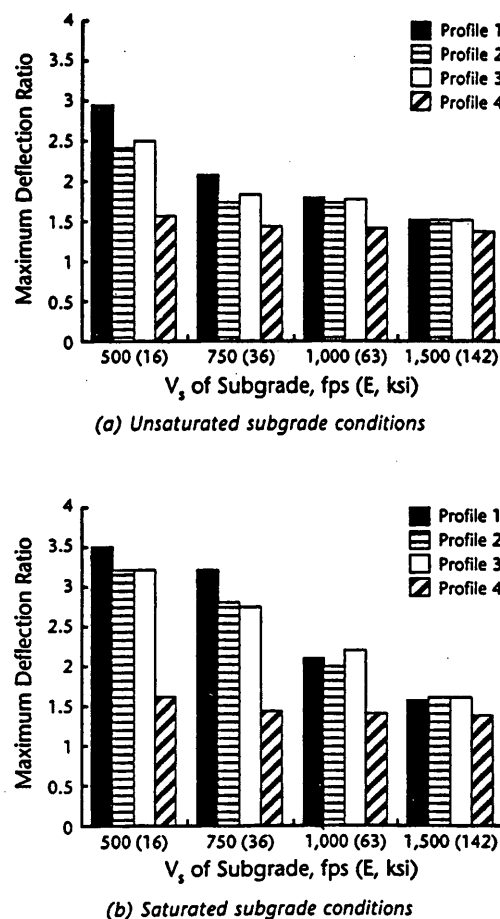
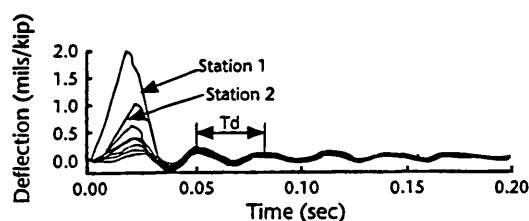


FIGURE 8 Maximum deflection ratios for FWD testing at the four pavement profiles with various subgrade stiffnesses.

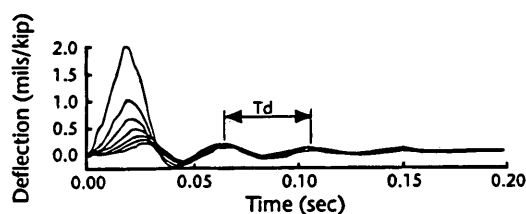
accurate than those obtained at sites with saturated subgrade conditions (4).

ESTIMATION OF DEPTH TO BEDROCK FROM FWD TEST

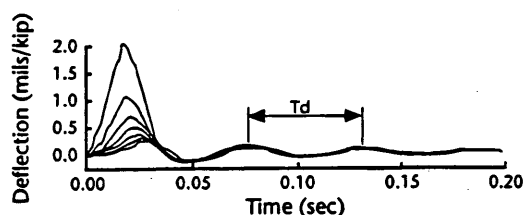
Chang et al. (2) developed a procedure for predicting the depth to bedrock based on the natural period of free vibrations of the pavement system immediately after FWD loading. Figure 9 illustrates the natural periods in the time-deflection records obtained from FWD tests with four shallow depths to bedrock at Profile 1



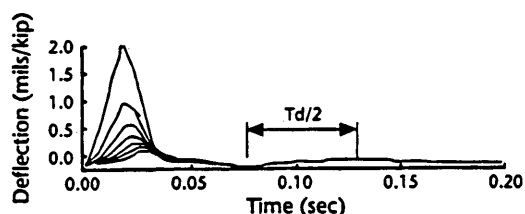
(a) Depth to bedrock = 5 feet



(b) Depth to bedrock = 7.5 feet



(c) Depth to bedrock = 10 feet



(d) Depth to bedrock = 20 feet

FIGURE 9 Deflection-time histories in FWD testing and damped natural periods (T_d) for Profile 1 with various depths to bedrock [V_s of subgrade = 500 fps (155 m/sec) and $E = 16$ ksi (0.11 MN/m²)].

for unsaturated subgrade conditions. It should be noted that a longer time interval (~ 0.12 to 0.20 sec) than normally recorded in FWD testing (0.06 sec) must be measured to record the free vibrations with confidence. FWD equipment is often set to record 60 msec of motion at 0.1-msec intervals. Increasing Δt to 0.2 or 0.4 msec would increase the duration without loss in accuracy. Care may be necessary to avoid drift but even this might be relatively unimportant.

Additional studies that are presented in this article have been conducted beyond those performed in Chang et al. (2). In these studies, various stiffnesses of the subgrade layer and different subgrade saturation conditions were examined. This work was performed to provide a more complete evaluation of the estimation of bedrock depth. Three different degrees of saturation for unsaturated subgrade conditions were simulated by using Poisson's ratio of 0.20, 0.33, and 0.40 representing dry, moist, and wet (but unsaturated) conditions, respectively, while keeping the shear wave velocity constant. The remaining material properties are the same as those in Table 1 (except for the compression-wave velocity). The material properties for the case of saturated subgrade conditions are shown in Table 2.

Four different shallow depths to bedrock of 1.5, 2.3, 3.1, and 6.1 m (5, 7.5, 10, and 20 ft, respectively) were studied. These depths were selected because the resonant depth to bedrock for the FWD test was always within 6 m (20 ft) of the pavement surface as shown in Figure 3, unless a very stiff subgrade [$V_s > 458$ m/sec (1500 fps)] was encountered.

Unsaturated Subgrade Conditions

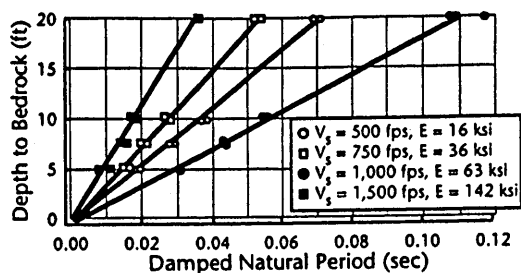
The depth to bedrock versus natural period, T_d , of the free vibrations for an unsaturated subgrade with Poisson's ratio of 0.20 is shown in Figure 10(a) and 10(b) for flexible and rigid pavements, respectively. The same set of plots for an unsaturated subgrade with Poisson's ratio of 0.33 is shown in Figure 10(c) and (d). These two sets of subgrade conditions can be considered to represent subgrades on the dry side of the optimum moisture content. For the case of the subgrade having Poisson's ratio equal to 0.40 (subgrade at or wet of the optimum moisture content), the linear relationship between depth to bedrock and the natural period of each profile is shown in Figure 11(a) and (b). There is a linear (or nearly linear) relationship between depth to bedrock and natural period for each stiffness and state of saturation of the subgrade in these figures. It should be noted that depth to bedrock is defined as the total depth from the top of the pavement to the top of the bedrock.

The equations of the straight lines for the flexible pavements with different Poisson's ratios (ν) of the subgrade can be combined into one equation as:

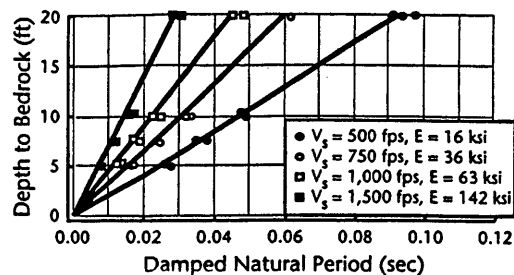
$$D_b = \frac{V_s T_d}{(\pi - 2.24 \nu)} \quad (5)$$

The equations for the rigid pavement with the subgrade having a range in Poisson's ratios (ν) can also be combined into one equation as:

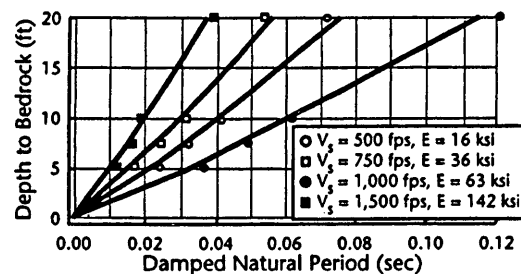
$$D_b = \frac{V_s T_d}{(\pi - 1.44 \nu)} \quad (6)$$



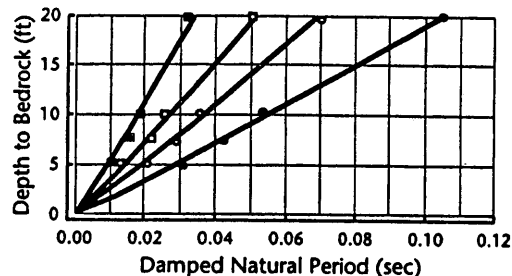
(a) Flexible pavements with Poisson's ratio of the unsaturated subgrade = 0.20



(c) Flexible pavements with Poisson's ratio of the unsaturated subgrade = 0.33

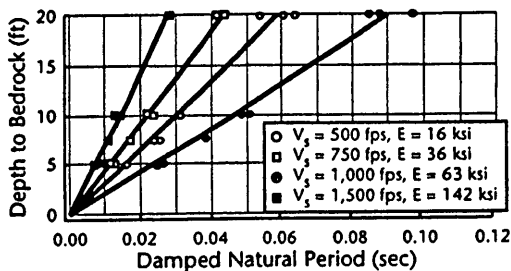


(b) Rigid pavements with Poisson's ratio of the unsaturated subgrade = 0.20

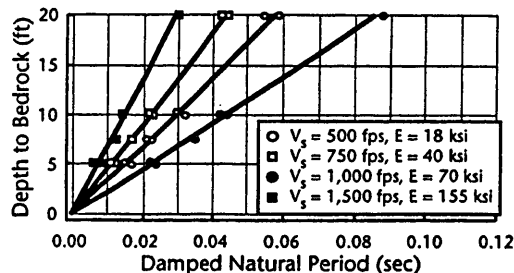


(d) Rigid pavements with Poisson's ratio of the unsaturated subgrade = 0.33

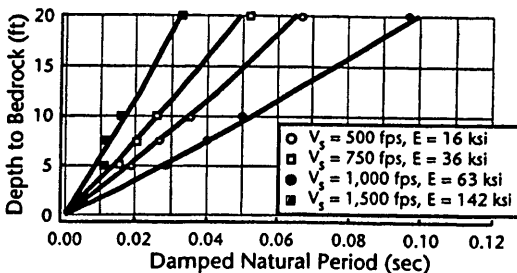
FIGURE 10 Depth to bedrock versus damped natural period for FWD testing of pavement systems with subgrades having low to medium degree of saturation.



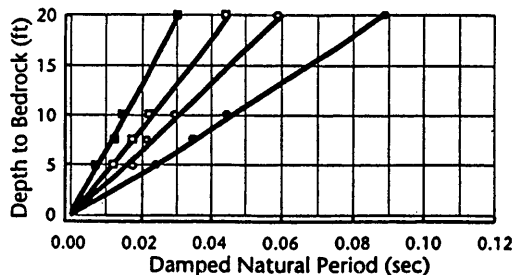
(a) Flexible pavements with Poisson's ratio of the unsaturated subgrade = 0.40



(c) Flexible pavements with a saturated subgrade



(b) Rigid pavements with Poisson's ratio of the unsaturated subgrade = 0.40



(d) Rigid pavements with a saturated subgrade

FIGURE 11 Depth to bedrock versus damped natural period for FWD testing of the pavement systems with nearly saturated and saturated subgrades.

In Equations 5 and 6, D_b and V_s can be expressed in any consistent set of units. Equations 5 and 6 can be expressed in terms of Young's modulus of the subgrade instead of its shear wave velocity assuming a value of the mass density as before. Using a density of 2 000 kg/m³ (or a unit weight of 110 pcf in British units) the expression substituting for V_s becomes:

$$V_s = \frac{\sqrt{E}}{63 \sqrt{1+\nu}} \left(\frac{\sqrt{E}}{2.6 \sqrt{1+\nu}} \right) \quad (7)$$

Saturated Subgrade Conditions

For saturated subgrade conditions, four different values for Poisson's ratios were used (0.495, 0.489, 0.479, and 0.451), which correspond to the four stiffnesses of the subgrade that result in the compression wave velocity equaling 1 525 m/sec (5,000 fps) (see Table 2). Figure 11(c) and 11(d) show the depth to bedrock versus natural period in this case. The equation for the flexible pavement is:

$$D_b = \frac{V_s T_d}{2.22} \quad (8)$$

The fitted equation for the rigid pavement can be expressed as:

$$D_b = \frac{V_s T_d}{2.31} \quad (9)$$

The expressions in terms of the Young's modulus can be obtained again using Equation 7.

ESTIMATION OF SUBGRADE STIFFNESS FROM FWD TESTS

To use the above equations, a good estimate of the stiffness of the subgrade is required. The stiffness of the subgrade can be estimated by in situ seismic testing, by dynamic laboratory tests on undisturbed samples, or possibly even by experience. However, a more convenient and accurate way to estimate subgrade stiffness was developed in this study. It was observed that one could measure the offset time (T_o) of the first pulse in the time-deflection records as shown in Figure 12. The offset time is related to the Rayleigh wave velocity of the subgrade. With the assumptions discussed in the following section, and knowing the distance between two measurement stations, the shear wave velocity can be determined.

There are several assumptions that must be made to use the offset time method. First, the subgrade should be able to be approximated as a uniform material. Second, the wavelength should be long enough so that the surface, base, and sub-base layers have little effect on the Rayleigh wave velocity. Generally, this means that the wavelength should be at least 10 times the total thickness of the surface, base, and sub-base layers for untreated bases and sub-bases (5). Third, the bedrock needs to be deep enough so that it has little effect on the Rayleigh wave velocity. This condition is usually met if the bedrock depth is greater than 0.5 times the Rayleigh wavelength in the subgrade. Fourth, it is assumed that near-field effects are small and that they can be ignored. Fifth, the first pulses of Stations 5 and 7 were used to measure the offset time because the deflections obtained at the stations away from the source should better represent the properties of the subgrade. Finally, the difference

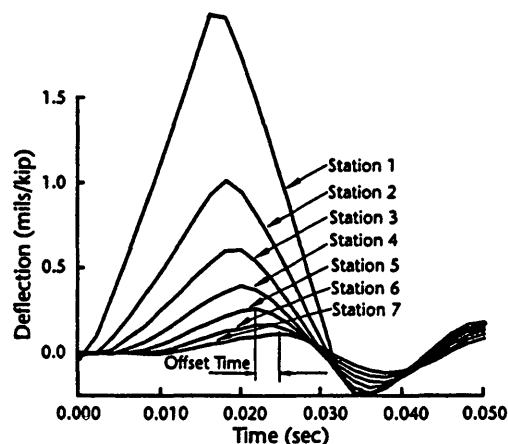


FIGURE 12 Offset time of the first pulses between Stations 5 and 7 for FWD testing at Profile 1 [V_s of subgrade = 500 fps (155 m/sec), $E = 16$ ksi (0.11 MN/m²) and depth to bedrock = 1.6 m (5 ft)].

between the Rayleigh wave velocity and the shear wave velocity is less than 10 percent if Poisson's ratio of the material is greater than 0.2 (3) so that it can be ignored. (Note: these assumptions may not apply in many test situations.)

Table 3 illustrates an example of comparisons between the estimated shear wave velocity of the subgrade and the actual shear wave velocity for Profile 1 with an unsaturated subgrade having a Poisson's ratio equal to 0.20. Good estimations of the shear wave velocity of the subgrade are made in the case of the softest subgrade. However, as the stiffness of the subgrade increases and as the stiffnesses and thicknesses of the upper pavement layers increase, there are cases in which estimations cannot be made. This happens because the first pulses in the deflection-time records were distorted and the peaks could not be determined.

SUMMARY

Analytical simulations of the FWD test were conducted using the computer program UTFWD (1,2). Four pavement profiles ranging from flexible to rigid were studied. The stiffness of the subgrade layer and the depth to bedrock (thickness of the subgrade) were the only parameters varied to simulate typical ranges in pavement systems that could be tested using the FWD.

Equations for estimating the resonant depth to bedrock, RD_b , were developed for both the flexible and rigid pavements. Depths to bedrock at which resonance effects may occur were found to vary from 1.7 to 6.2 m (5.5–20 ft), depending on the stiffness of the subgrade. Saturated subgrade conditions gave the same trend in resonant depth to bedrock with subgrade stiffness.

Equations for estimating the depth to bedrock based on the natural period of free vibrations of the pavement system immediately after FWD load application have been presented. In these equations, effects of stiffness and degree of subgrade saturation were taken into account. One important aspect in applying these equations is that complete time histories recorded for 0.12 sec or longer are desired.

TABLE 3 Estimated Shear Wave Velocity of Subgrade from Offset Time of the First Pulses for FWD Testing at Profile 1

Units: Vs (fps), E (ksi)

Depth to Bedrock (ft)	Actual Vs of Subgrade (E of Subgrade)					
	500 (16)			750 (36)		
	To (sec)	Estimated Vs (fps)	Error** (%)	To (sec)	Estimated Vs (fps)	Error (%)
5	0.0035	571	14%	na***	na	na
7.5	0.0039	513	3%	0.0028	714	-5%
10	0.0040	500	0%	0.0029	690	-8%
20	0.0042	476	-5%	0.0030	667	-11%

Depth to Bedrock (ft)	Actual Vs of Subgrade (E of Subgrade)					
	1000 (62)			1500 (142)		
	To (sec)	Estimated Vs (fps)	Error (%)	To (sec)	Estimated Vs (fps)	Error (%)
5	na	na	na	na	na	na
7.5	na	na	na	na	na	na
10	0.0020	1000	0%	na	na	na
20	0.0021	952	-5%	0.0013	1538	3%

* Poisson's Ratio of Subgrade equals 0.20.

** Error=[(Estimated Vs/Actual Vs)-1]*100%

*** Offset Time is not available because the first pulses in FWD tests are distorted.

A method to estimate the stiffness of the subgrade from the offset time of the first pulses of the deflection-time records at two measurement stations in the FWD tests has been proposed. The shear wave velocity of the subgrade can be estimated dividing the offset time into the distance between these two stations. At present, it seems that this approach is more appropriate in cases in which the stiffness of the subgrade is soft to moderate, $V_s = 155$ to 233 m/sec ($V_s = 500$ – 750 fps) [$E = 0.11$ to 0.25 MN/m² ($E = 16$ – 36 ksi)] and the bedrock depth is 3.1 m (10 ft) or more.

ACKNOWLEDGMENTS

We thank the Texas Department of Transportation for partial support of the study under project 1175. The support of the Advanced Technology Program of the State of Texas is also sincerely appreciated.

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Publication of this paper sponsored by Committee on Soil and Rock Properties.