Nondestructively Delineating Changes in Modulus Profiles of Secondary Roads

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To load-zone roads properly, mechanisms involved in the deterioration of pavements must be understood and monitored. The state of practice in nondestructively evaluating pavement systems is limited to determining changes in modulus profiles. For secondary roads, deflection basin methods [such as falling-weight deflectometer (FWD) and Dynaflect] are most effective in determining moduli of subgrades and are not as sensitive to moduli of the surface and base layers. On the other hand, the Spectral-Analysis-of-Surface-Waves (SASW) method is quite sensitive to moduli in the upper layers. In addition, the SASW method has the advantage of allowing the pavement system to be divided into numerous layers, say 10 to 15 in the upper 3 ft, so that detailed profiles can be determined. With this resolution, it is possible to delineate changes in the modulus profile from one measurement to the next. To illustrate the use of the SASW method on secondary roads, two sites were tested to determine the possible reasons for one section rutting and the other not. The rutted section was found to have layers with lower moduli or in which moduli appeared to be decreasing and hence possibly causing deterioration of the section. Also, the softening effect of rain on these pavements was studied. Softening occurred mainly in the upper portion of the subgrade. The FWD device was also used to determine moduli of the two sections. Moduli from the FWD tests are substantially lower than those from the SASW tests in the base layers mainly because of nonlinear behavior created during FWD testing. However, moduli of the subgrades are quite similar because of the linear behavior of this material in both types of tests. Deflection basins based on moduli of SASW tests are also compared with the FWD deflection basins. If the nonlinear effects are considered, the deflection basins based on moduli evaluated by the SASW tests compare well with those measured by the FWD device.

The Spectral-Analysis-of-Surface-Waves (SASW) method was used at two sites on Farm-to-Market (FM) Road 2001 located near Buda, Texas. This method was first used in March 1986 to determine the variation of in situ Young's modulus with depth. The objective of testing was to determine the sensitivity of the SASW method to the degree of deterioration of the pavement; surface observations of one test site revealed no deterioration, and at the second test site some deterioration was manifested in surface rutting. The two sites are representative of several similar comparative tests performed on different road sections in central Texas.

A second series of SASW tests was performed at the two sites on FM-2001 in June 1986. The objective of these tests was to determine the effect of heavy rains, which had occurred during the week before the tests, on the moduli of the different pavement layers. The effect of the rains was evaluated by comparing moduli from the second series of tests with those determined from the tests in March. A series of falling-weight deflectometer (FWD) tests was also performed on this day for comparison purposes. After completion of data reduction, the two sites were cored to verify the reported layer thicknesses.

SASW tests are performed at low strain levels where pavement and soil layers behave linearly. An algorithm has been developed to model in an approximate manner the effect on the deflection basin of the nonlinear behavior in the different pavement layers. With this model, deflection basins based on the small-strain moduli determined by the SASW tests but approximately modified to account for nonlinear behavior were computed. These nonlinear deflection basins were then compared with deflections measured by the FWD device. The two basins compare well when nonlinear behavior is taken into account.

Results of this study are presented herein, along with brief background information on the SASW method and an explanation of the test procedures and data analyses. A detailed description of the testing technique and the theoretical background can be found elsewhere (*1-3*).

SPECTRAL-ANALYSIS-OF-SURFACE-WAVES TESTING

General Background

The SASW method is a type of seismic testing that was developed for determining shear wave velocity and elastic shear modulus profiles at soil sites and elastic Young's modulus profiles at pavement sites (*1-3*). The SASW method is a nondestructive method in which both the source and the receivers are located on the ground surface. The source is simply a transient vertical impact that generates a group of surface waves of various frequencies that the medium transmits. Two vertical receivers located on the surface monitor the propagation of surface wave energy. By analysis of the phase information of the cross power spectrum for each frequency determined between the two receivers, phase velocity, shear wave velocity, and elastic moduli are determined.

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The key points in SASW testing are generation and measurement of surface waves (Rayleigh waves). Rayleigh wave velocity (V_R) is constant in a homogeneous half-space and independent of frequency. Each frequency (f) has a corresponding wavelength (L_R) according to

$$V_R = f \times L_R \quad (1)$$

Rayleigh wave and shear wave velocities are related by Poisson's ratio. In an isotropic elastic half-space, the ratio of Rayleigh wave to shear wave velocity increases as Poisson's ratio increases. The change in this ratio is small, and it can be assumed that the ratio is approximately equal to 0.90 without introducing an error large than 5 percent.

If the stiffness of a site varies with depth, the velocity of the Rayleigh wave (R-wave) will vary with frequency. The variation of R-wave velocity with frequency (wavelength) is called dispersion, and a plot of surface wave velocity versus wavelength is called a dispersion curve. The dispersion curve is developed from phase information of the cross power spectrum. This information provides the relative phase between two signals (two-channel recorder) at each frequency in the range of frequencies excited in the SASW test. For a travel time equal to one period of the wave, the phase difference is 360 degrees. Thus, for each frequency, the travel time between receivers can be calculated by

$$t(f) = \phi(f)/(360 \times f) \quad (2)$$

where

f = frequency,
$t(f)$ = travel time for a given frequency, and
$\phi(f)$ = phase difference in degrees for a given frequency.

The distance between the receivers (X) is a known parameter. Therefore, R-wave velocity at a given frequency [$V_R(f)$] is simply calculated by

$$V_R(f) = X/t(f) \quad (3)$$

and the corresponding wavelength of the R-wave is equal to

$$L_R(f) = V_R(f)/f \quad (4)$$

By repeating the procedure outlined by Equations 2-4 for every frequency, the R-wave velocity corresponding to each wavelength is evaluated and the dispersion curve is determined.

Rayleigh wave velocities determined by this method are not actual velocities of the layer but are apparent R-wave velocities (known as phase velocities). Existence of a layer with high or low velocity at the surface of the medium affects measurement of the velocities of the underlying layers. Therefore, a method for evaluation of shear wave velocities from phase velocities (apparent surface wave velocity) is necessary in SASW testing.

Inversion of the dispersion curve, or (in short) inversion, is the procedure of determining the shear wave velocity profile from the dispersion curve. Inversion consists of determination of the depth of each layer and the actual shear wave velocity of each layer from the apparent R-wave velocity versus wavelength information.

The inversion process used herein is based on a modified version of Thomson's (*4*) and Haskell's (*5*) matrix solution for elastic surface waves in a layered solid medium. To simplify the process of inversion, some assumptions were made: (*a*) the layers are horizontal, (*b*) the velocity of each layer is constant, and (*c*) the layers are homogeneous and linearly elastic. Assumptions *a* and *b* are quite reasonable for most pavement systems within the top 3 to 5 ft where the vast majority of SASW data is collected. Also, Assumption *b* (constant velocity within a layer) does not limit the variability in the complete modulus profile because numerous layers (5 to 10) can be used to represent the final profile. Assumption *c* is also quite realistic for the small-strain seismic waves used in these tests. Nonlinear behavior in the pavement system can then be accounted for by combining field (linear) and laboratory (nonlinear) testing as discussed in the section on nonlinear moduli.

The inversion process is an iterative one in which a shear wave velocity profile is assumed and a theoretical dispersion curve is constructed. The experimental and theoretical dispersion curves are compared and necessary changes are made in the assumed shear wave velocity profile until the two curves (experimental and theoretical dispersion curves) match within a reasonable tolerance. Detailed discussions of the inversion process are given elsewhere (*2, 3, 6*).

When the shear wave velocities have been determined, the following formulas are used to calculate shear and Young's moduli:

$$G = \rho \times V_s^2 \quad (5)$$

$$E = 2G(1 + \nu) \quad (6)$$

where

G = shear modulus,
E = Young's modulus,
ρ = mass density, and
ν = Poisson's ratio.

Field Procedure

The general configuration of the source, receivers, and recording equipment is shown in Figure 1*a*. Accelerometers were used as receivers for close receiver spacings (4 ft and less), and geophones with a natural frequency of 4.5 Hz were used as receivers for greater spacings. This was done to optimize recording of the wave passage; that is, accelerometers give more output over a wide frequency range at closer receiver spacings where high frequencies are present and geophones give more output at larger receiver spacings where low frequencies are excited.

The common receivers midpoint (CRMP) geometry (*1*) was used for testing. With this geometry the two receivers were moved away from an imaginary centerline midway between the receivers at an equal pace, and the source was

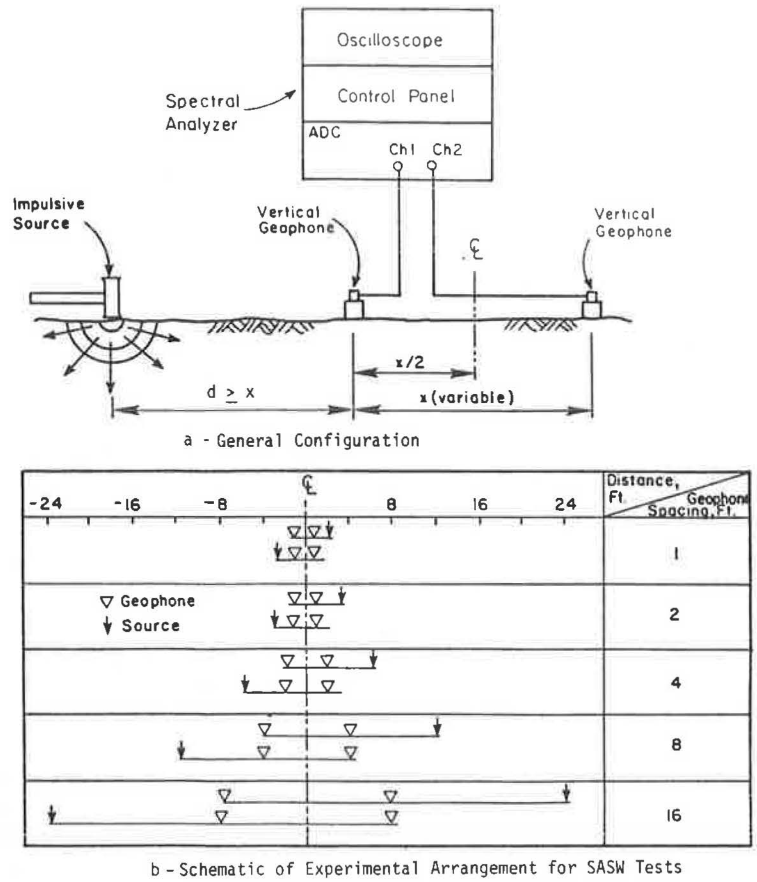


FIGURE 1 Field procedure used in SASW testing.

moved so that the distance between the source and the near receiver was equal to or greater than the distance between the two receivers. In addition, the location of the source was reversed for each receiver spacing so that forward and reverse profiles were run. This testing sequence is shown in Figure 1*b*. Distances between receivers of 0.5, 1, 2, 4, and 8 ft were used at each site.

Different sources were used. For close receiver spacings, a 4-oz hammer was used. For greater distances, sledgehammers were employed.

The recording device was a Hewlett-Packard 3562A Fourier spectral analyzer. A Fourier analyzer is a digital oscilloscope that by means of a microprocessor attached to it has the ability to perform directly in either the time or frequency domain. Fourier analysis is a power tool in decomposition of complicated waveforms, and testing could not be performed without such an analysis.

It should be emphasized that the field operation can be fully automated. Two different avenues are currently being pursued to automate field testing. The first consists of installing a powerful minicomputer and data-acquistion system in a van to control the impact system and recording and manipulating the receivers' output. The second is to modify an FWD system for this purpose. Both methods will reduce testing time to a few minutes per site.

EVALUATION OF NONLINEAR MODULI

There are two basic approaches used today to evaluate moduli of pavement systems in the field. The first approach is to employ high-intensity loads in an attempt to evaluate the nonlinear behavior of the pavement. Elastic theory is then used to backcalculate the modulus profile. The advantage of this approach is that an equivalent nonlinear modulus of the pavement may be determined. However, if these moduli are used to determine the stresses and strains in the pavement system, substantial errors may occur because the modulus profile is approximated with only three or four equivalent moduli and, hence, may only be appropriate for calculating surface displacements under loads similar to those used to evaluate the equivalent moduli. The FWD is a good example of this testing approach.

The second approach is to determine elastic moduli in situ and to perform laboratory tests on representative samples to define the decrease of modulus with increasing strain (and to some extent with the change in stress state). Then, by incorporating these two (laboratory and field) results, the actual nonlinear behavior of the pavement system can be determined for any load level. More than 30 years of research in earthquake engineering have shown that the second approach is quite realistic. The SASW testing method falls into this second category of testing.

As an example, a model used to describe the relationship between the modulus and strain for geotechnical materials is shown in Figure 2. The model is based on numerous resonant column tests on granular and cohesive soil samples (*7*). The model is quite representative of the behavior of subgrades, granular bases, and subbases. It can be seen that the effects of

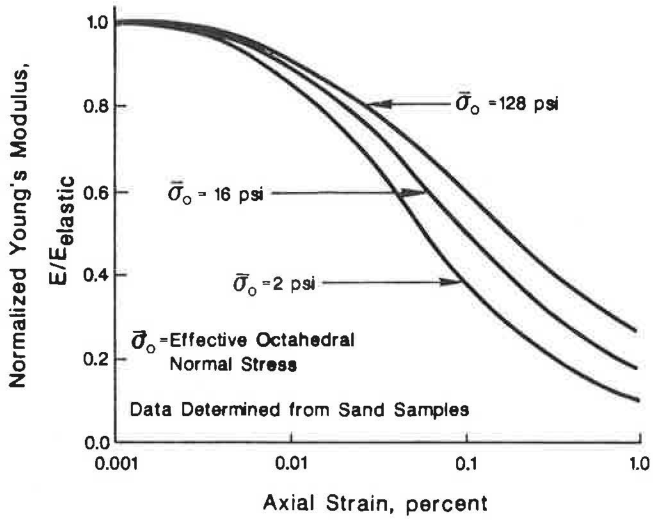


FIGURE 2 Nonlinear model used for determining equivalent linear moduli of geotechnical materials.

both normal strain and normal stress on moduli are considered in the model. Several important points can be deduced from Figure 2. First, as the octahedral normal stress increases in a layer, the material become stiffer (at a constant strain). Second, there is a threshold strain level below which the soil behaves elastically. This threshold level is slightly above 0.001 percent. Third, an increase in strain above the threshold level results in a reduction in the value of the modulus of the material and, hence, nonlinear behavior.

One important conclusion that can be drawn from the material behavior illustrated in Figure 2 is that results from the SASW and FWD tests should be quite similar if FWD tests are performed at low levels of load that only create small strains in all parts of the pavement system and if both testing techniques are analyzed correctly.

DESCRIPTION OF SITES

Two sites were tested in March 1986. The two sites were located 0.8 mi apart. Site 1 was at Milepost 2 on FM-2001 in the outer wheelpath of the westbound lane. Site 2 was about 0.8 mi to the west of the first site. The SASW test was carried out in the outer wheelpath of the eastbound lane at Site 2.

Visually, the pavement at Site 1 was in excellent condition, and no cracking or depression could be located. At Site 2 cracks were visible, and the pavement was depressed and rutted. The depth of rutting at this site was approximately 1 in.

Material profiles of the two sites were reported to be the same (on the basis of construction drawings). At each site, the first inch of material consisted of an asphalt-treated surface course. About 10 in. of granular base underlay the surface course. The base was, in turn, underlain by a clayey subgrade.

Coring the two sites (in February 1987) revealed the following profiles: At Site 1, the asphalt layer was 1.25 in. thick. The base consisted of two sublayers. The upper sublayer of the base, with a thickness of 6 to 7 in., consisted of good granular base materials. However, the lower sublayer of the base, 3 to 4 in. thick, had a very high clay content. The clay in this base layer was yellowish and appeared to have been placed with the granular base. The subgrade, which can be categorized as Houston black clay, was encountered at approximately 11 in. below the surface. The asphalt layer at Site 2 was approximately 1.5 in. thick. The base consisted of three distinct sublayers. The first sublayer was 2.5 in. thick and consisted of good base material similar to the material found in the upper sublayer of the base at Site 1. The second sublayer, which extended to a depth of 8 in. from the surface, was not as competent as the first sublayer and contained some clay. The third sublayer of the base was about 3 in. thick and consisted of a base material with a high content of yellowish clay. The Houston black clay subgrade was encountered at a depth of 11 in. below the surface.

MODULUS PROFILES FROM SASW TESTS

Before-Rain Tests

Dispersion curves from SASW testing at Sites 1 and 2 are shown in Figure 3. Typical spectral analysis functions measured at these sites for one receiver spacing are shown in Figure 4. The quality of the data collected in the field was quite good (as judged by the writers based on much previous experience).

Shear wave velocity profiles determined from inversion of the dispersion curves are given in Column 4 of Tables 1 and 2 for Sites 1 and 2, respectively. Seven layers with a total thickness of about 23 in. over an eighth layer (a half-space) were used in the inversion process at each site (Figure 5*a*). The thicknesses of the layers ranged from 1 in. near the surface to 12 in. in the upper portion of the subgrade. The number of layers was limited to eight because the intent was to use an elastic layered program (LAYER8 program) to obtain the theoretical deflections from the SASW modulus profiles. Program LAYER8 can only analyze a layered system with no more than eight layers.

As a first approximation in the inversion process, layer thicknesses reported in the construction records were used. The assumed layering subsequently changed during the final

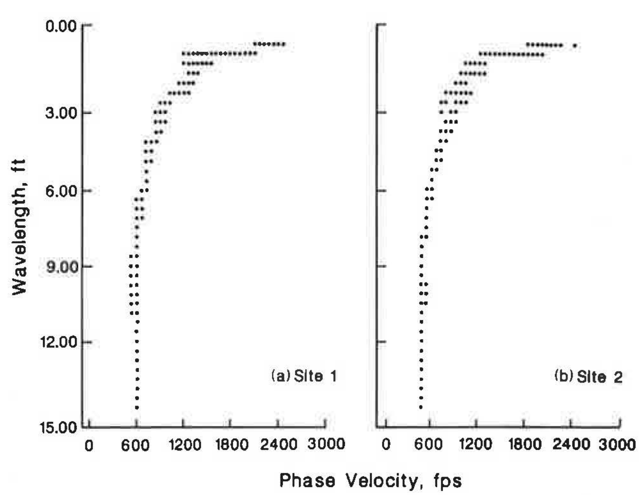


FIGURE 3 Dispersion curves obtained from SASW tests before rain.

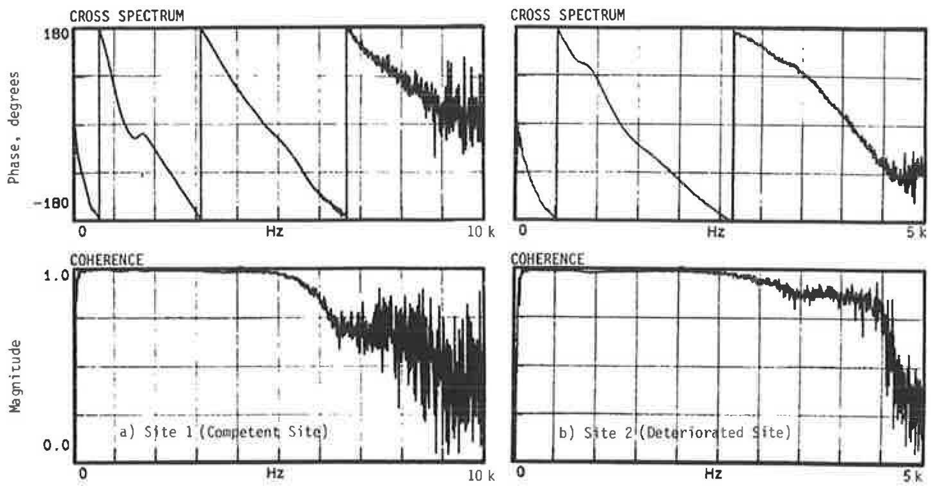


FIGURE 4 Spectral analysis measurements from surface wave testing before rain.

TABLE 1 VARIATION OF SHEAR WAVE VELOCITY AND YOUNG'S MODULUS WITH DEPTH FROM SASW TESTS AT SITE 1 (competent pavement)

			Before Rain		After Rain	
Layer Number	Layer Thickness in.	Layer Depth[1] in.	Shear Wave Velocity fps	Young's Modulus[2] ksi	Shear Wave Velocity fps	Young's Modulus[2] ksi
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	0.96	0.48	3063	592.0	3036	592.0
2	1.20	1.56	3007	570.6	3007	570.6
3	2.40	3.36	1470	136.4	1470	136.4
4	2.40	5.76	1465	135.4	1465	135.4
5	2.04	7.98	946	56.5	946	56.5
6	2.04	10.02	744	34.9	744	34.9
7	12.00	17.04	586	21.6	527	19.1
8	H-S[3]	---	608	23.4	547	20.6

[1] Depth to the midheight of the layer

[2] Based on an assumed total unit weight of 110 pcf for all materials

[3] Denotes Half-Space

TABLE 2 VARIATION OF SHEAR WAVE VELOCITY AND YOUNG'S MODULUS WITH DEPTH FROM SASW TESTS AT SITE 2 (deteriorated pavement)

			Before Rain		After Rain	
Layer Number	Layer Thickness in.	Layer Depth[1] in.	Shear Wave Velocity fps	Young's Modulus[2] ksi	Shear Wave Velocity fps	Young's Modulus[2] ksi
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	0.96	0.48	2968	555.9	2968	555.9
2	1.20	1.56	2577	419.0	2577	419.0
3	1.20	2.76	1144	82.6	1144	82.6
4	2.40	4.56	1114	78.3	1114	78.3
5	2.40	6.96	1064	71.5	1064	71.5
6	3.00	9.66	698	30.7	663	27.7
7	12.00	17.16	540	18.4	462	14.6
8	H-S[3]	---	563	20.4	496	16.9

[1] Depth to the midheight of the layer

[2] Based on an assumed total unit weight of 110 pcf for all materials

[3] Denotes Half-Space

inversion process as discussed later. In the inversion process, values of Poisson's ratio of 0.33, 0.33, and 0.45 were assumed for the asphaltic cement (AC), base, and subgrade, respectively. Misestimation of Poisson's ratio has a minimal effect on the shear wave velocities obtained by the inversion process (*2*).

On the basis of the shear wave velocity profiles given in Tables 1 and 2, Young's moduli at different depths were calculated using Equations 5 and 6. The resulting Young's modulus profiles are shown in Figures 5*c* and 5*e* and given in Tables 1 and 2 for Sites 1 and 2, respectively. To calculate Young's moduli, a total unit weight of 110 pcf was assumed for all layers. Because the objective of this paper is to determine the percentage difference in moduli of the different layers, misassumption of the total unit weight does not affect the generality of the discussion presented in the next section.

Analysis of Before-Rain Tests

The thicknesses of the different layers at Sites 1 and 2 estimated from the modulus profiles compare only generally with the layering based on the construction plans. Therefore it was decided to core the sites to determine the actual pavement profiles. For both sites, the total thicknesses of the base and wearing course agree well with the construction plans (11 in.). However, the differences in the materials used in the base were evident from coring. The layering found from

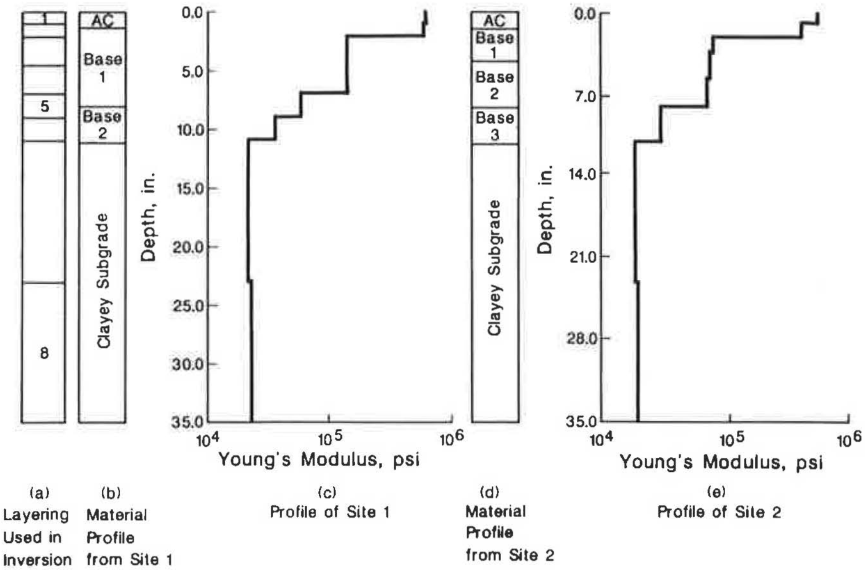


FIGURE 5 Composite profiles of the two sites for before-rain conditions.

coring supports the variation in the base moduli obtained from the SASW tests. The bottom of the base material exhibits a stiffness that is closer to the stiffness of the subgrade. This is probably because the bottom sublayer of the base has a high clay content. Also, the modulus of the top sublayer of the base is close to that of the surface layer. This may be due to a combination of reasons. First, additional compactive effort was applied to the base, especially the upper portion, when the asphaltic surface was placed. Second, because the top two layers at each site were each assumed to be about 1 in. thick in the inversion process, the stiffness of the lower of these two layers is an average of the asphalt and base moduli because the thickness of the asphalt layer was actually more than 1 in. but less than 2 in.

Because the SASW method is sensitive to deterioration of pavement, a brief discussion of the differences and similarities of the two dispersion curves is in order. The dispersion curves from Sites 1 and 2 are compared in Figure 6 on expanded scales. In Figure 6*a*, the two dispersion curves are compared over a range in wavelengths from 0 to 3 ft. It can be seen that the two curves follow each other quite well in the ranges of wavelengths less than 0.5 to 2.5 ft. However, in the range of wavelengths of 0.5 to 2.5 ft, the dispersion curve from Site 2 indicates phase velocities on the order of 10 to 25 percent less than those of Site 1. A very approximate rule of thumb [The following discussion is presented for a better understanding of the role of dispersion curves and is not meant to imply that it can be used as a replacement for the inversion process.] suggests that the effective depth of sampling is equal to one-third (*8*) to one-half (*9*) of the wavelength and that shear wave velocity is equal to 110 percent of phase velocity. If this rule is applied, it can be concluded that the shear wave velocity of the base material of the first site (competent pavement) is approximately 10 to 25 percent higher than that of the second site (deteriorated pavement). A 25 percent difference in shear wave velocities corresponds to a difference in moduli of about 56 percent.

In Figure 6*b* the dispersion curves in the range of wavelengths of 3 to 10 ft are shown. It can be seen that the two curves are quite similar with a difference of about 10 percent. In other words, on the basis of the rule of thumb, the subgrade of Site 1 has a shear wave velocity that is approximately 10 percent higher than the velocity of Site 2. The two curves show the same trend and velocities below wavelengths of 10 ft and, therefore, are not included.

The shear wave velocity profiles of the two sites (after the true inversion process, not rule of thumb) are compared in Table 3. The material profile from coring is included in Figure 5. It can be seen that the shear wave velocity of the asphalt layer is approximately equal for both sites. Parts of the base material (the first inch) show nearly the same stiffness as the AC layer, particularly at Site 1. Below the upper stiff portion of the base, the shear wave velocity of the base material at Site 1 is about 30 percent greater than that at Site 2. The bottom 2 in. of the base materials at both sites exhibit stiffnesses closer to those of the subgrades than the bases. The shear wave velocities of the subgrades are nearly the same, differing only by about 8 percent. A significant point is that the layering in the base materials observed from coring is well reflected in these profiles.

Young's moduli obtained from the two sites are compared in Figure 7 and Table 3. Based on Equations 5 and 6, the differences in the values of Young's moduli from the two sites should be the square of the differences in the shear wave velocities. Therefore the base materials are about 70 percent stiffer at Site 1 than at Site 2, and the subgrade is about 15 percent stiffer at Site 1 than at Site 2. One possible reason for the deterioration of the pavement at Site 2 is the softer base materials.

After-Rain Tests

Dispersion curves from SASW testing at Sites 1 and 2 in June are shown in Figure 8. The quality of the data collected in the field was again good.

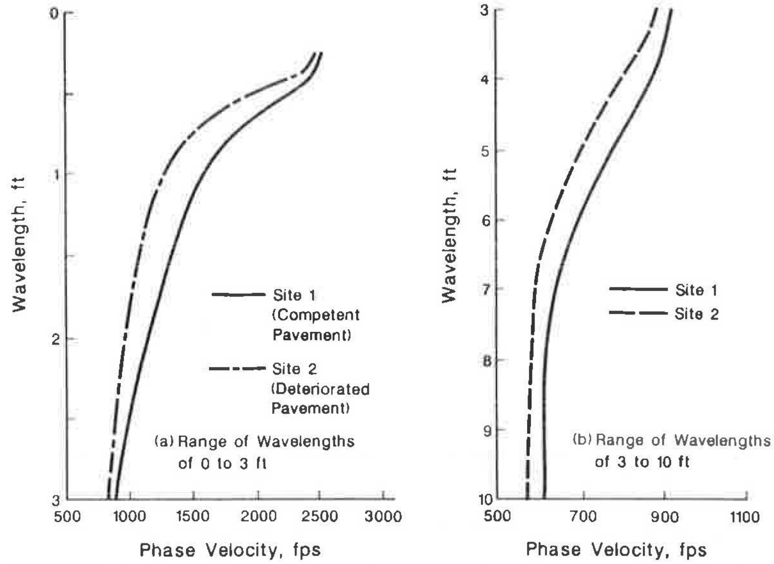


FIGURE 6 Comparison of dispersion curves from Sites 1 and 2 before rain.

TABLE 3 COMPARISON OF SHEAR WAVE VELOCITIES AND YOUNG'S MODULI DETERMINED BY SASW TESTS AT SITES 1 AND 2 BEFORE RAIN

Layer No.	Depth in.	Shear Wave Velocity fps		Velocity Difference percent	Young's Modulus ksi		Modulus Difference percent
		SITE 1	SITE 2	(3-4)/4	SITE 1	SITE 2	(6-7)/7
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	0.5	3063	2968	3.2	592.0	555.9	6.5
2	1.6	3007	2577	16.7	570.6	419.0	36.2
3	3.4	1470	1144	28.5	136.4	82.6	65.1
4	5.8	1465	1114	31.5	135.4	78.3	72.9
5	8.0	946	1064	-11.1	56.5	71.5	-21.0
6	10.0	744	698	6.6	34.9	30.7	13.7
7	17.0	586	540	8.5	21.6	18.4	17.4
8	H-S	608	563	8.0	23.4	20.4	17.0

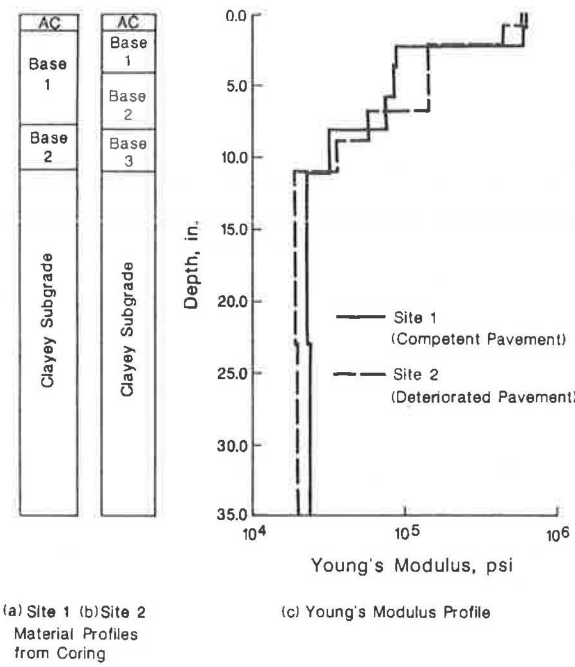


FIGURE 7 Comparison of Young's modulus profiles from Sites 1 and 2 before rain.

Shear wave velocity profiles determined from inversion of the dispersion curves are given in Column 6 of Tables 1 and 2 for Sites 1 and 2, respectively. As was done in the first series of tests, the same eight layers were used in the inversion process at each site to evaluate the stiffness in the top 35 in. of the pavements. It should be noted that, as the starting point in the inversion process, shear wave velocity profiles obtained from the first series of tests at these sites (March) were used.

Based on the shear wave velocity profiles, Young's moduli at different depths were calculated using Equations 5 and 6. The resulting Young's modulus profiles are shown in Figure 9 and given in Tables 1 and 2 for Sites 1 and 2, respectively.

Comparison of Moduli for Before- and After-Rain Conditions

Moduli obtained from the first series of tests when the pavement was not subjected to heavy rain and moduli from the second series, which was performed immediately after a week of heavy rain, are compared. Dispersion curves obtained from before- and after-rain tests are compared in Figure 10 for Sites 1 and 2. At both sites, the upper parts of the dispersion curves (say to a wavelength of 3 ft) are almost identical. This is an indication of similarity between the shear wave velocities of the two tests (i.e., before- and after-rain tests) to a depth of approximately 1 to 1.5 ft. In other words, the stiffnesses of AC and base layers were not affected by the rainfall. Phase velocities from after-rain tests were about 10 percent smaller than those from before-rain tests for wavelengths greater than about 4 ft, which means that the rain softened the subgrade.

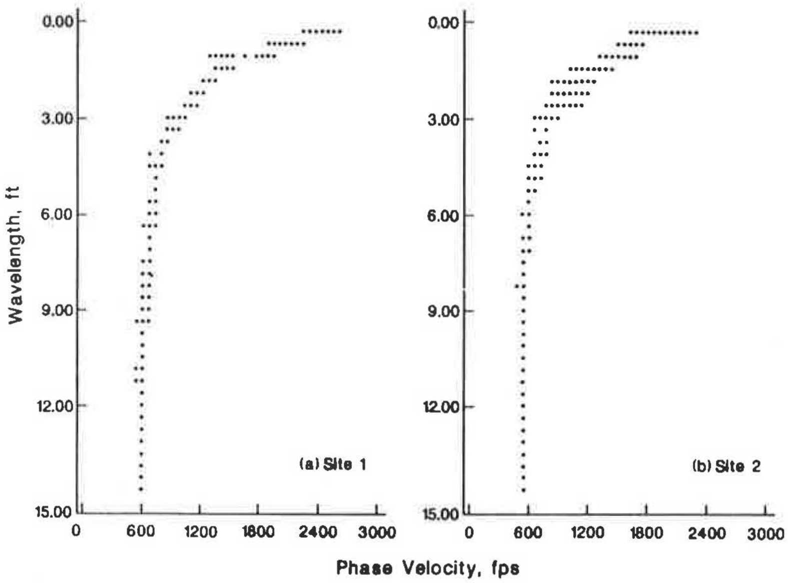


FIGURE 8 Dispersion curves obtained from SASW tests after rain.

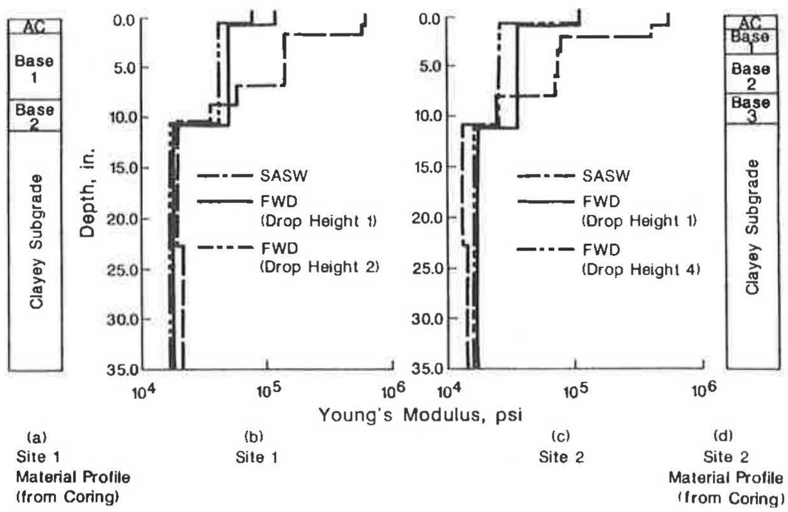


FIGURE 9 Composite profiles from SASW and FWD tests at the two sites after rain.

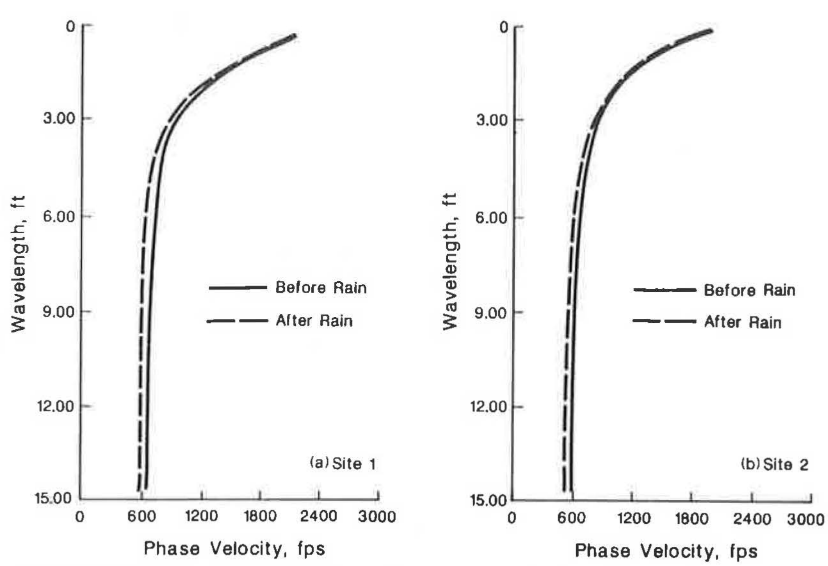


FIGURE 10 Comparison of dispersion curves obtained by SASW tests for before- and after-rain conditions.

Young's moduli from the two sets of tests performed at Sites 1 and 2 are compared in Figure 11. The moduli of the first six layers of Site 1 and the first five layers of Site 2 are the same for both series of tests. However, moduli of the last two layers of Site 1 are approximately 20 percent softer. Moduli of the the last three layers of Site 2 are as much as 30 percent softer.

Comparison of Moduli from SASW and FWD Tests

Unfortunately, FWD tests were not carried out during the first series of tests in March 1986. Therefore only FWD results for Sites 1 and 2 can be compared for the after-rain series. Measured deflection basins are given in Table 4 for nominal drop loads of 5 and 15 kips. The FWD sensors were spaced 1 ft apart. For both loads, the measured basins are quite sensitive to the overall differences between the pavement sites; that is, the deflections at Site 2 are larger than those at Site 1. However, at both sites, deflections measured at the farther sensors (5 and 6) are nearly the same, which indicates that subgrade moduli at the two sites are quite similar; the subgrade modulus at Site 2 is slightly smaller than that at Site 1. This is the same relationship that was found by the SASW tests.

Modulus profiles of the two sites backcalculated by basin fitting are given in Tables 5 and 6 for Sites 1 and 2, respectively. These modulus profiles appear to indicate that the cause of pavement deterioration at Site 2 (in comparison with Site 1) is the softness of the base layer, which represents the primary difference between the sites. Modulus profiles from the SASW tests for both the before- and after-rain conditions show the same trend.

Moduli determined from nominal loads of 5 and 15 kips are compared in Tables 5 and 6 for both sites. Moduli of base materials obtained from the smaller load are larger than those from the larger load at both sites. This is due to nonlinear behavior of the base material at the higher load as discussed subsequently.

Moduli obtained by the SASW and FWD tests are compared in Figure 9 and Table 5 for Sites 1 and 2, respectively. Subgrade moduli compare well in both cases. However, moduli of the base and AC layers are much higher from the SASW tests. This difference in base moduli is due mainly to nonlinear behavior caused by the high load levels in the FWD tests. It is difficult to comment on the difference in moduli of the AC layer, primarily because of the insensitivity of the FWD method to thin surface layers.

As a further comparison of moduli from FWD and SASW tests, moduli determined from the SASW tests were input to program LAYER8, a modified version of the NLAYER program (*10*), to determine a theoretical deflection basin for each site. These theoretical basins are denoted as SASW (L) and are compared with the measured ones [denoted as FWD (M)] in Table 4. Predicted deflections at the last three sensors compare well with the measured ones. The deflections of the first three sensors are smaller for the theoretical basins [SASW (L)] determined from moduli measured by the SASW method. The reason for the favorable comparisons between the deflections of Sensors 4-6 is most probably that these sensors monitor essentially linear behavior.

To investigate in an approximate fashion the effect of nonlinearity on the moduli of different layers, program LAYER8 (*10*) was modified to perform an equivalent linear analysis. In an equivalent linear analysis, a modulus at a known strain level is assumed. The strain is calculated on the basis of the assumed modulus and compared with the assumed strain. If the two strains are within an acceptable tolerance (say 10 percent), the assumed modulus is selected as the so-called "nonlinear" modulus or "equivalent linear" modulus. Otherwise a new modulus based on the average of the calculated and the assumed strains is assumed, and the process is repeated until the strains converge.

The model used to describe the relationship between the modulus and strain is shown in Figure 2. The model is quite suitable for subgrade and granular base materials. This model was assigned to all layers in the pavement systems (including the asphalt layer). Because nonlinearity occurs

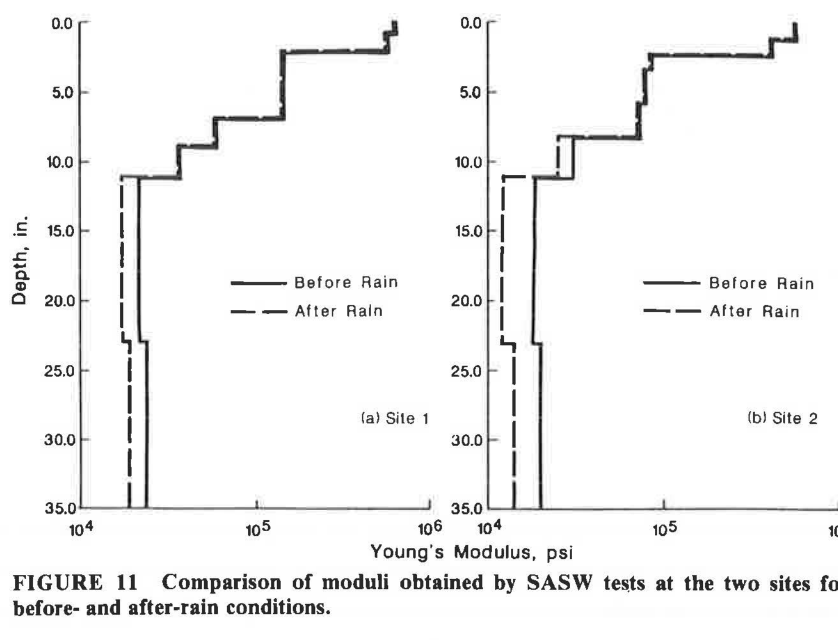


FIGURE 11 Comparison of moduli obtained by SASW tests at the two sites for before- and after-rain conditions.

TABLE 4 COMPARISON OF MEASURED AND THEORETICAL DEFLECTION BASINS FROM FWD AND SASW TESTS

Nominal Load, lb	Site	Deflection Basin Source	Deflections, mils Sensor Number 1	2	3	4	5	6	Mean Square Error, Percent
5000	1	FWD (M)*	13.3	6.4	3.2	1.9	1.5	1.1	---
		FWD (B)*	13.1	5.7	3.1	2.0	1.5	1.1	2.1
		SASW (L)**	8.0	4.7	2.8	1.8	1.3	1.0	8.3
		SASW (NL)**	11.8	6.0	3.3	1.8	1.3	1.0	2.4
	2	FWD (M)	16.2	7.9	3.4	2.1	1.5	1.2	---
		FWD (B)	16.2	6.1	3.2	2.0	1.5	1.2	4.0
		SASW (L)	11.0	6.1	3.5	2.2	1.6	1.2	6.6
		SASW (NL)	17.9	8.1	4.2	2.2	1.6	1.2	4.3
15000	1	FWD (M)	48.4	25.4	12.3	7.1	4.9	3.7	---
		FWD (B)	48.2	19.0	10.0	6.5	4.7	3.7	5.5
		SASW (L)	25.4	10.9	8.9	5.7	4.1	3.2	13.9
		SASW (NL)	50.8	23.4	11.2	6.8	4.7	3.2	3.2
	2	FWD (M)	63.1	34.2	13.1	6.6	4.9	4.0	---
		FWD (B)	63.5	21.1	10.6	6.9	5.0	4.0	7.2
		SASW (L)	35.1	19.5	11.8	7.0	5.1	3.8	10.5
		SASW (NL)	86.0	33.0	14.7	8.4	5.8	3.8	8.5

*M = Measured, B = Deflections from the fitted basin
**L = Linear, NL = Equivalent linear

TABLE 5 COMPARISON OF MODULI OBTAINED BY SASW AND FWD TESTS AT SITE 1

Layer	Layer Thickness, in.	Young's Modulus, ksi SASW	FWD 5 kips	FWD 15 kips	Difference, percent 5 kips (3)-(4)/(3)	Difference, percent 15 kips (3)-(5)/(3)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
AC	1	592	115	75	80.6	87.3
Base	1.2	571	50	40	91.2	93.0
	2.4	136			63.2	70.6
	2.4	135			63.0	70.4
	2.0	56.5			16.0	29.2
	2.0	34.9			-30.2	-14.6
Subgrade	12	19.1	18	17.5	5.8	8.4
	-	20.6			12.6	15.0

TABLE 6 COMPARISON OF MODULI OBTAINED BY SASW AND FWD TESTS AT SITE 2

Layer	Layer Thickness, in.	Young's Modulus, ksi SASW	FWD 5 kips	FWD 15 kips	Difference, percent 5 kips (3)-(4)/(3)	Difference, percent 15 kips (3)-(5)/(3)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
AC	1	556	110	110	80.2	80.2
Base	1.2	419	35.0	25.0	91.6	94.0
	1.2	82.6			57.6	69.7
	2.4	78.3			55.3	68.1
	2.4	71.5			51.0	65.0
	3	27.7			-26.4	-9.8
Subgrade	12	14.6	17.2	16.5	17.8	13.0
	-	16.9			-1.8	2.4

mainly in the base and subgrade layers, use of this approximation should not result in significant error.

To determine the nonlinear deflection basin, the modulus profile determined from the SASW tests was used as initial input to modified LAYER8. In addition, the profile at each site was divided into eight layers, as shown in Figure 5*a*, with the eighth layer extending to infinity. The algorithm was then used to calculate the equivalent linear modulus at the middle of each layer at radial intervals of 3 in., starting at the centerline of the loaded area and proceeding to a distance at which the initial and equivalent linear moduli were identical (i.e., the moduli were in the linear range, which occurred at distances of from 30 to 50 in. for these tests). It should be mentioned that, because radial variation in moduli cannot be accounted for in LAYER8, some approximation had to be introduced in the calculation of strains and stresses. This approximation was as follows: After the nonlinear modulus profile at each radial distance from the source was obtained, the original version of program LAYER8 (i.e., the linear elastic version) was used to calculate surface deflections. This was done by assigning the equivalent linear modulus profile obtained from the previous calculations at the sensor location of interest and then calculating the surface deflection at that sensor. Therefore, to obtain the deflection basin at each site, program LAYER8 was used six different times (for six sensors) with six different modulus profiles. [The effect of this approximation is under study with a finite-element analysis. However, any adverse effect of this approximation should be most important to deflections at the second and third sensors.]

Deflections obtained as described previously are included in Table 4 as the nonlinear SASW results [denoted as SASW

(NL)]. Deflections based on small-strain (linear) SASW moduli [denoted as SASW (L)] and the FWD basin-fitting procedure [denoted as FWD (B)] are also included in Table 4. It can be seen that the nonlinear SASW deflection basins follow the measured ones favorably, except for the nominal load of 15 kips at the deteriorated site where the deflection of the first sensor is overestimated by about 30 percent.

The percentage differences between the three sets of theoretical deflections [FWD (B), SASW (L), and SASW (NL)] and the measured FWD deflections at each site for the two nominal loads (5 and 15 kips) are shown in Figure 12. The average mean square error of each theoretical basin relative to the measured deflection basin is included in Table 4. The average mean square errors from the basin fitting of the FWD data and nonlinear SASW modulus profiles are quite close, indicating that (at least statistically) both basins (i.e., from basin fitting and from equivalent linear modulus profiles) are almost equally representative of the measured deflection basins. This favorable comparison is a good indication of the accuracy of the moduli obtained by the SASW method, with an added advantage that the nonlinear performance of the pavement system for any given load can be easily estimated. As an example, the ratios of equivalent linear moduli (for a nominal load of 5 kips) to elastic moduli at Site 1 (both obtained from SASW tests) are shown in Figure 13. Variation in the ratios with radial distance from the load for five points within the pavement system are shown. As can be seen in Figure 13, the asphalt layer and the deepest subgrade point do not exhibit much nonlinear behavior (assuming the asphalt layer has the relative change in modulus with strain shown in Figure 2). Also, it can be seen that, from the center of the loaded area out to normalized radial distances of about 1.5, the base material close to the base-subgrade interface is critical because it exhibits the highest amount of nonlinearity. From that point on, the subgrade exhibits a larger amount of nonlinearity. At normalized radial distances of 5 or more, all layers exhibit linear behavior.

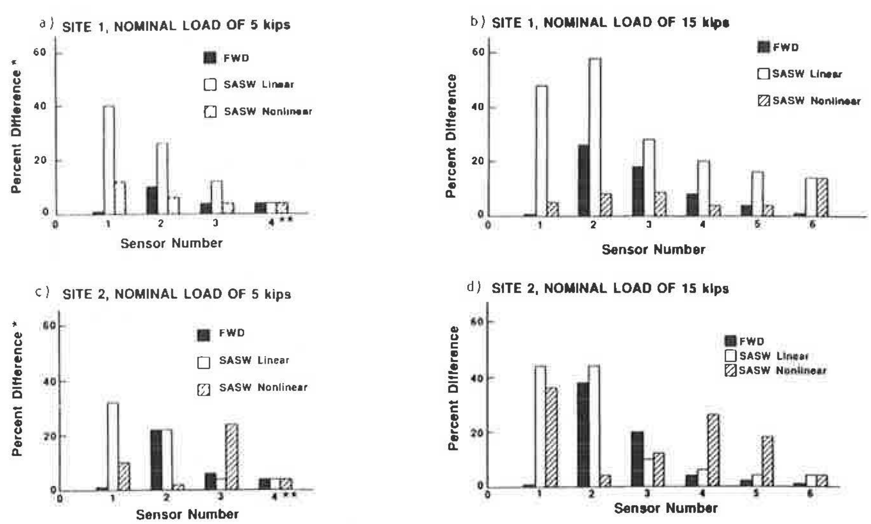
SUMMARY AND CONCLUSIONS

One problem facing the engineer is evaluating the integrity of existing pavements. When secondary roads are load zoned, the mechanisms that cause failure of the pavement should be well understood in terms of delineation of possible zones of weakness or seasonal change. It is demonstrated that the SASW method can be used to evaluate effectively possible causes of deterioration in pavement sections. The SASW method is a powerful tool because of the numerous layers that can be used to create fine, detailed resolution in the modulus profile.

In network-level studies, the dispersion curves obtained from SASW tests can be used effectively to determine the approximate location of weak zones. However, the testing technique should be automated to achieve this goal. At the project level, modulus profiles of the pavement system from SASW testing can be determined in detail. At this time, a typical SASW test takes about 30 min to perform in the field and about 2 hr to reduce in the office. Both aspects of field testing and in-house data reduction are being automated to reduce the testing time and data reduction to several minutes for future network-level studies.

To illustrate the use of the SASW method on secondary roads, two sites were tested to evaluate the accuracy of layering determined from the modulus profiles, determine possible reasons for rutting at one of the sites, and evaluate any changes in the modulus profiles resulting from heavy rains. The following results were found.

1. Layering estimated from the modulus profiles at both sites differed somewhat from layers shown on the construction plans. Coring of the sites after completion of the SASW tests substantiated that the layering estimated from the SASW tests was more representative of the actual material profiles than were the construction plans.



* Corresponds to the absolute value of: (measured FWD deflection minus deflection predicted from modulus profile) divided by deflection predicted from modulus profile

**Sensors 5 and 6 are not shown because in all cases differences were less than the accuracy of sensors (5/± .1 mil)

FIGURE 12 Comparison of deflection basins measured by FWD tests and those backcalculated from modulus profiles estimated from SASW and FWD tests.

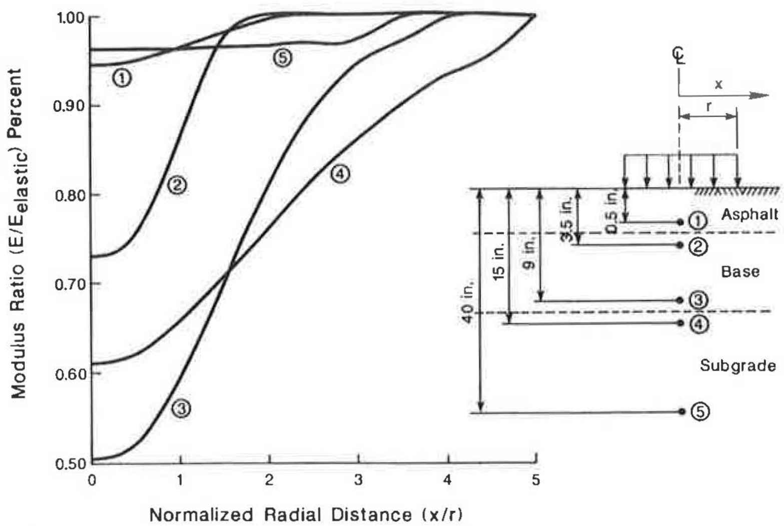


FIGURE 13 Distribution of reduction of modulus with depth and radial distance at Site 1 for FWD testing with a nominal load of 5 kips.

2. The reason for rutting at one of the two sites appears to be the base layer, which was about 70 percent softer at the rutted site, although a 15 percent softer subgrade at the rutted site may also have contributed to the surface rutting.
3. The effect of heavy rain at the two sites was primarily a softening of the subgrade. This softening turned out to be slightly more severe at the deteriorated site.

In addition to the SASW tests, FWD tests were performed at the two sites after the heavy rains for comparison purposes. These tests showed the following results:

1. Modulus profiles determined from the FWD and SASW tests indicate that both test methods show the relative softness of the base material at the rutted site.
2. Moduli determined from the FWD and SASW tests differed significantly for the base layer primarily because of the different strain levels associated with the two testing methods. Moduli of the base layer from the FWD tests were significantly lower, with moduli decreasing with increasing load levels as expected for nonlinear behavior.
3. Calculated deflections based on the SASW modulus profiles compare well with deflections measured by the FWD tests for the last three sensors. At the first three sensor locations, measured deflections are significantly greater than those predicted with the (linear) SASW modulus profiles. If nonlinear effects are taken into account, equivalent linear moduli from the SASW tests produce deflection basins that statistically follow the measured FWD deflection basins as well as those of the fitted FWD basin.

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