Comparison of Theoretical and In Situ Behaviors of a Flexible Pavement Section

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The use of nondestructive testing (NDT) devices has provided pavement engineers with a simple method for determining pavement layer moduli. Moduli obtained with the NDT devices are used in mechanistic pavement design. Therefore, it is important to determine accurately the elastic moduli of pavement systems. Through a case study, some concerns with the deflection-based NDT of pavements are examined. The main objectives were (a) to establish the accuracy of a backcalculation methodology, (b) to determine the closeness of the theoretical and measured deformations within a pavement cross section, and (c) to assess the impact of the established accuracies on the design of pavement sections. These items are discussed through an example from one instrumented site. The instrumentation and algorithm used to determine in situ deflections are briefly described. A total of seven state-of-the-art NDT devices were employed. Particular attention was devoted to the effects of the load-induced nonlinearity associated with the large magnitude of loads imparted by the NDT devices to the pavement. On the basis of the case study, it was found that some differences in the deflections predicted the use of backcalculated moduli and measured deflections. It was also shown that load-induced nonlinearity may be one of the reasons for the poor match between the measured and theoretical deflection basins.

Mechanistic pavement design has been used more frequently in recent years than in earlier years. The backbone of mechanistic pavement design is nondestructive testing (NDT). To confirm the validity of mechanistic models and NDT methods, it has become important to monitor the behavior and performance of pavements under actual loads. As such, much effort has been focused on instrumenting pavements. One response parameter usually measured is the deflection within the body of the pavement. Multidepth deflectometer (*1*) or geophone units (*2*) can be used to obtain this information.

One site was instrumented with geophones and tested with seven NDT devices. The main objective of this study was to determine how well layered elastic theory in conjunction with NDT devices can predict the behavior of pavements. To achieve this objective, several steps were taken. The first step was to determine the effects that the load level and the sensor locations might have on backcalculated moduli. For each NDT device, and each load level, up to 19 deflections were available, 12 within the pavement section and the rest on the surface. Eight combinations of deflections were used in backcalculation. On the basis of these results, a thorough discussion of the effects that the load level, location, and number of deflection sensors and type of NDT device may have on backcalculated moduli was carried out.

The second goal was to determine how well the deformations, stresses, and strains are determined. A comprehensive comparison of stresses and strains from various types of NDT devices used is also included in this paper.

BACKGROUND

Seven commercially available dynamic deflection measuring devices, including four falling weight deflectometer (FWD) devices and three vibratory loading devices (Dynaflect, Road Rater, WES 16-kip), were used in this study. These devices are well described by Bentsen et al. (*3*) and others.

An experimental study of these seven nondestructive testing devices by Bentsen et al. (*3*) shows that each device is capable of producing and collecting nondestructive pavement test data consistently and reliably. However, they also indicated that, as with any type of test equipment, the data should not be considered error-free. Care should be taken when field data are employed for pavement evaluation purposes, such as in the determination of allowable loads, overlay thicknesses, and layer moduli.

A side-by-side evaluation of deflection measuring equipment was carried out by Hudson et al. (*4*). Three FWD devices, a Road Rater, and a Dynaflect were evaluated. The report indicated that the results from these devices are satisfactory.

TESTING PROGRAM AND DATA COLLECTION

Description of Site

The site was located in the apron of Sheppard Air Force Base in Wichita Falls, Tex. As shown in Figure 1, the profile consisted of a 7-in. asphaltic concrete (AC) surface layer, a base layer of about 20 in. of granular material, underlain by a sandy clay subgrade.

Instrumentation

The installation process is described in detail by Nazarian et al. (*5*). Four two-dimensional geophone units were installed at the site. Each geophone unit consisted of two geophones (one horizontal and one vertical). The two were carefully placed inside a 2-in.-long, 4-in. outside-diameter polyvinyl chloride (PVC) pipe. The geophones were then covered with

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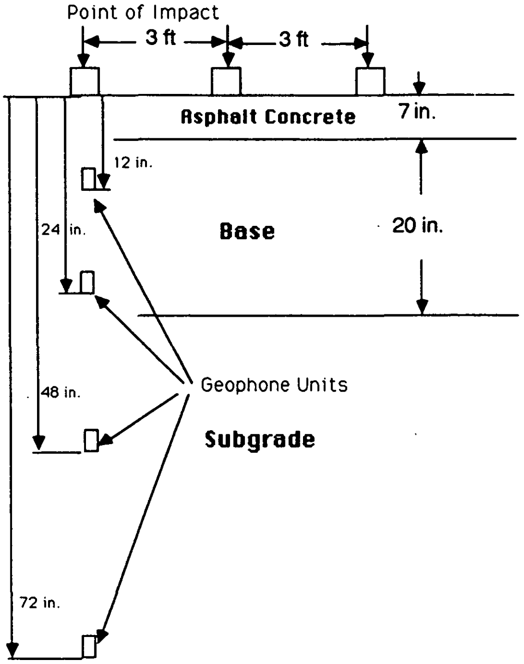


FIGURE 1 Schematic of testing program.

an epoxy-resin mix using the PVC pipe as the mold to eliminate the possibility of corrosion and moisture damage.

Testing Program

A schematic of the testing program is illustrated in Figure 1. The loading mechanism of the NDT device was placed on top of the borehole containing the four geophone units. Data were collected with both vertical and horizontal receivers. The NDT device was then moved about 3 ft and the testing was repeated. Once again, the device was moved colinearly another 3 ft (total of 6 ft from the borehole) and tests were repeated. The voltage output of the geophones was monitored, captured, and stored using two Hewlett-Packard two-channel spectral analyzers.

Determination of Field Deflections

The procedures adopted for determining the deflections from the geophone records are described here. The data reduction procedure is described in detail in Tandon and Nazarian (*2*). A frequency-domain solution is employed. Any function in the time domain can be easily expressed by a limited number of harmonic functions, if Fourier transform is utilized.

Shown in Figure 2 are the actual vertical displacement time histories sensed by the vertical geophones caused by a 25-kip load applied with an FWD. All figures are plotted to the same scale to demonstrate the variation in amplitude with distance from the point of impact. It can be seen that the deflections decrease rapidly with both radial and horizontal distance.

DETERMINATION OF MODULI

Moduli obtained from the backcalculation process are included in Table 1 for all load levels applied by all devices. The average absolute differences between the theoretical and measured basins after completion of deflection basin fitting vary between 0.9 and 5.7 percent, with an average of about 2 to 3 percent. Therefore, the matching has been done in a reasonable fashion.

Asphalt Layer

The thickness of the asphalt layer was 7 in. Therefore, theoretically speaking, one should be able to determine reasonably well the modulus of this layer.

The modulus of the asphalt layer varies from a minimum of 220 ksi to a maximum of about 1,080 ksi (619 ksi, if the result from the Road Rater is considered as an outlier). Overall, the coefficient of variation is equal to about 47 percent (29 percent without a modulus from the Road Rater).

Base Layer

As indicated before, the base was about 20 in. thick. Theoretically, the modulus of such a thick layer should be determined with no difficulty at all. Practically speaking, assuming a constant modulus for this layer may not be appropriate. The load-induced nonlinearity and method of construction might have caused significant heterogeneity in the base layer. The modulus varies between a minimum of 23 ksi and a maximum of approximately 63 ksi. The average modulus is about 37 ksi with a standard deviation of about 10 ksi.

Nonlinear behavior in terms of reduction in modulus with increase in load levels is not evident from the falling weight devices except for the KUAB device. However, for the vibratory devices the effects of nonlinearity is (at least) qualitatively evident. The Dynaflect, which applies 1 kip of load, has the highest modulus. Also for the WES vibrator, the modulus values increase as the load decreases.

Subgrade Layer

The modulus of the subgrade determined from deflection basins measured by various devices falls in a narrow range. The minimum and maximum values are approximately equal to 14 ksi and 20 ksi, respectively. The average modulus is about 16.5 ksi with a standard deviation of 1.4 ksi. The coefficient of variation is relatively small, about 8.5 percent; this indicates that the modulus of the subgrade layer, unlike the other two pavement layers, is more or less independent of the testing device.

Moduli from Alternative Sensor Configurations

Practically speaking, only surface sensors should be used in routine pavement evaluation. However, to understand the behavior of pavements under large loads and to determine

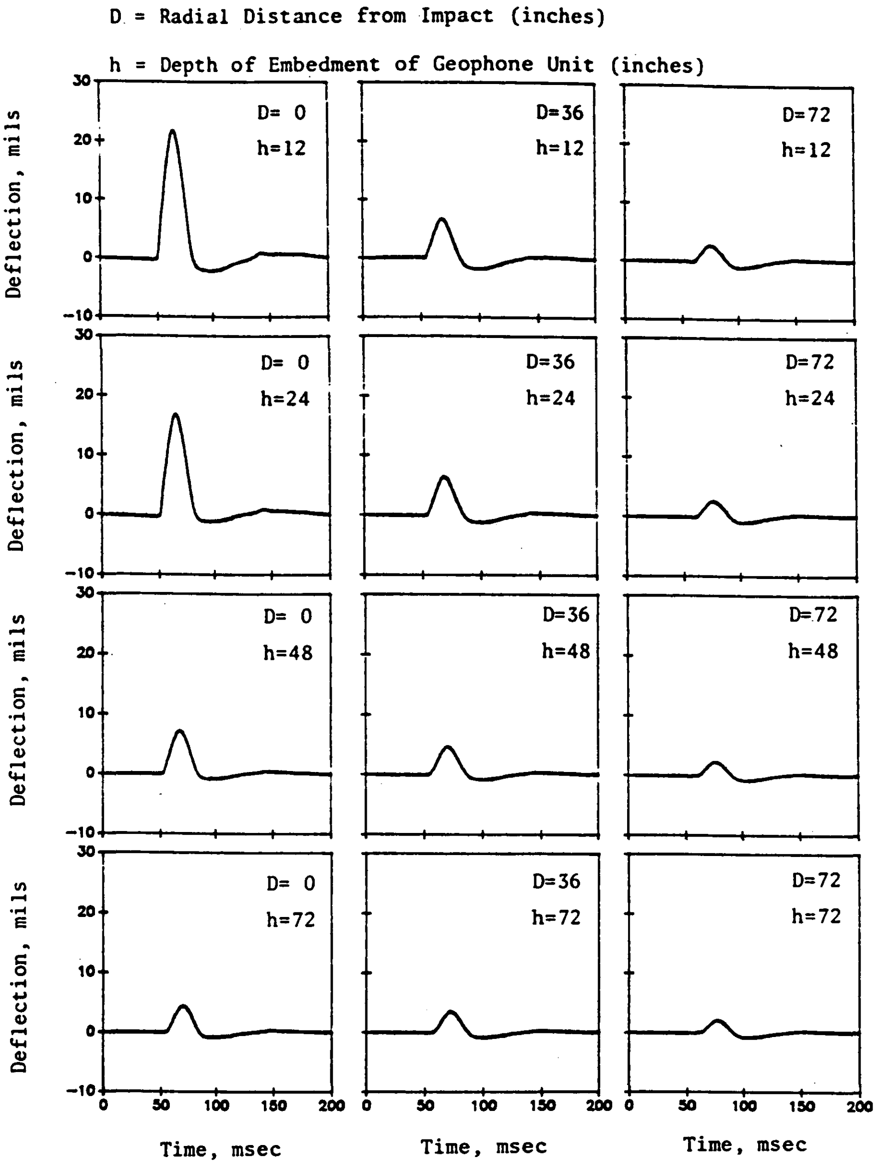


FIGURE 2 Vertical displacement time histories from embedded geophone units caused by effects of FWD.

the limitations of the NDT tests, it is desirable to determine moduli from other sensors embedded in the pavement. Eight different data sets were used for backcalculation, as shown in Figure 3. Data Set 1 corresponds to the surface deflections. Data Sets 2 through 5 contain only three deflection sensors. Each set represents the three sensor readings obtained at a given depth. Data Sets 6 through 8 correspond to a rather comprehensive set of deflections, as reflected in the figure.

Effects of Load-Induced Nonlinearity

To study the effect of load-induced nonlinearity on backcalculated moduli, deflection data from various depths were used to determine the moduli of various layers. Data Sets 2 through 5, described in Figure 3, correspond to this exercise.

The backcalculated moduli for the base and subgrade for each data set are shown in Figure 4 for one FWD. The results from six other NDT devices are presented in Chai (*6*). For the sake of brevity, they are not represented here. However, the conclusions drawn here are more or less applicable to other devices.

In the backcalculation methodology, the modulus of the AC layer was fixed as 400 ksi. This was a good average based on the backcalculation of the surface deflections. The reason for not backcalculating the modulus of AC layer was the scarcity of deflection data in some cases.

Strain contour lines for a 6-in.-radius plate used by several NDT devices are shown in Figure 5. The computer program BISAR was used to determine the strains and stresses. The average moduli obtained from all NDT devices using the surface deflections were input in to the program (see Table 1).

TABLE 1 Moduli Backcalculated from Surface Deflection Bowls Using Program BISDEF

Device, Load	Modulus of Asphalt (psi)	Modulus of Base (psi)	Modulus of Subgrade (psi)	Avg. Abs. Difference (Percent)
Dynatest 25 k	260,756	30,882	16,142	5.0
Dynatest 15 k	221,778	33,881	16,548	4.7
Dynatest 10 k	249,457	33,860	17,735	4.7
KUAB 15 k	496,736	32,494	18,379	1.0
KUAB 10 k	414,194	36,848	17,766	0.9
KUAB 5 k	341,198	41,816	16,745	1.2
Heavy FWD 45 k	468,549	22,647	14,109	5.7
Heavy FWD 25 k	333,651	23,794	16,121	4.4
Heavy FWD 15 k	353,214	23,202	16,560	2.8
Phoenix 20 k	364,974	33,719	16,792	4.5
Phoenix 15 k	343,811	34,303	16,990	4.5
Dynaflect 1 k	287,299	63,141	15,564	2.8
Road Rater 5 k	1080,891	35,809	19,669	1.8
WES 15 k	442,919	39,537	14,295	5.2
WES 10 k	517,562	46,221	14,953	5.2
WES 5 k	619,770	54,623	15,793	5.2
AVG	424,797	36,788	16,501	-
STD	198,894	10,531	1,388	-
COV	46.8	28.6	8.4	-

Deflection Data Set	Sensors Utilized
1	Surface Sensors only
2	11, 12, 13
3	21, 22, 23
4	31, 32, 33
5	41, 42, 43
6	01, 02, 03, 11, 12, 13, 21, 22, 23, 31, 32, 33, 41, 42, 43
7	11, 12, 13, 21, 22, 23, 31, 32, 33, 41, 42, 43
8	Surface Sensor, 11, 12, 13, 21, 22, 23, 31, 32, 33, 41, 42, 43

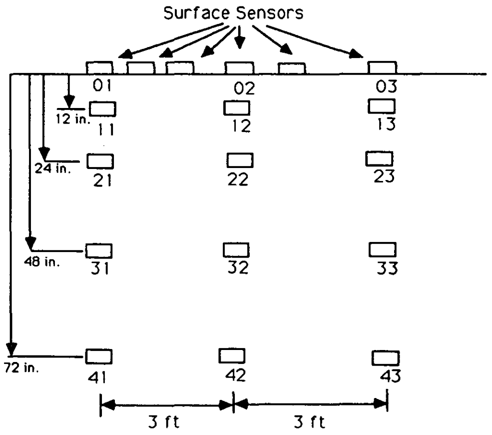


FIGURE 3 Deflection data sets used in backcalculation.

It is understood that utilizing a linear-elastic program for determining the contour levels may introduce some inaccuracies in the upcoming discussions. However, as demonstrated by Chai (*6*), the regions in which stresses and strains are relatively large are rather localized and limited to a small region close to the point of impact. Therefore, for regions several diameters away from the point of impact, the nonlinear and linear solutions should be similar.

To facilitate the upcoming discussion, for strain levels below 0.003 percent, it is assumed that the materials behave linearly. This region is called linear region. Any region in which the strain level lies between 0.003 and 0.01 percent is called a quasi-linear region. In this strain range, the reduction in moduli is typically less than 15 percent. The nonlinear region is the region in which the strains are greater than 0.01 percent. These ranges are selected on the basis of the terminology used in the earthquake geotechnical engineering.

In Table 2, the influence of load-induced nonlinearity as a function of the location of sensors is demonstrated. It can be seen that the sensors at a radial distance of zero are in either the nonlinear or quasi-linear range, and the sensors at radial distances of 72 in. are all in the linear range. One interesting point is that, for the sensors at radial distances of 36 in., deflections from Data Set 4 (depth of 48 in.) experience more load-induced nonlinearity than those from Data Set 3 (depth of 24 in.). This is because the sensors at the depth of 24 in. are embedded in the stiff base layer; whereas the sensors at a depth of 48 in. are located in the relatively soft subgrade soil. This matter is well reflected in the modulus of base layer (Figure 4).

The backcalculated moduli from this series is shown in Figure 4. From Data Set 3, one obtains smaller moduli than from Data Set 2. This is true for all three load levels. On the other hand, the modulus of the subgrade layer increases as the data sets from deeper strata are used.

For any given data set, moduli of the base and subgrade increase as the load level decreases. The increase in the modulus of the base under the lowest load is significantly higher for Data Sets 2 through 4. The reason for this matter is not known at this time.

Although not shown here, for data sets in which all three sensors are outside the nonlinear region, the average absolute difference is small (less than 3.5 percent); whereas, as soon as one or two of the sensors are located in the nonlinear range, a good basin fitting cannot be achieved. In the latter cases, the minimum value for the average absolute difference is about 7.8 percent. This can at least partially describe why in many pavement sections, the basic fitting cannot be carried out satisfactorily.

Effects of Increased Number of Sensors

Typically, in the nondestructive evaluation of pavement deflections, from four to seven surface sensors are used to backcalculate the moduli of two to four layers. In the program BISDEF, the least-squares criterion is used to evaluate the closeness of the theoretical deflections to the measured ones. Typically, when the least-squares criterion is used, the more overcompensated the problem is (i.e., the larger the number

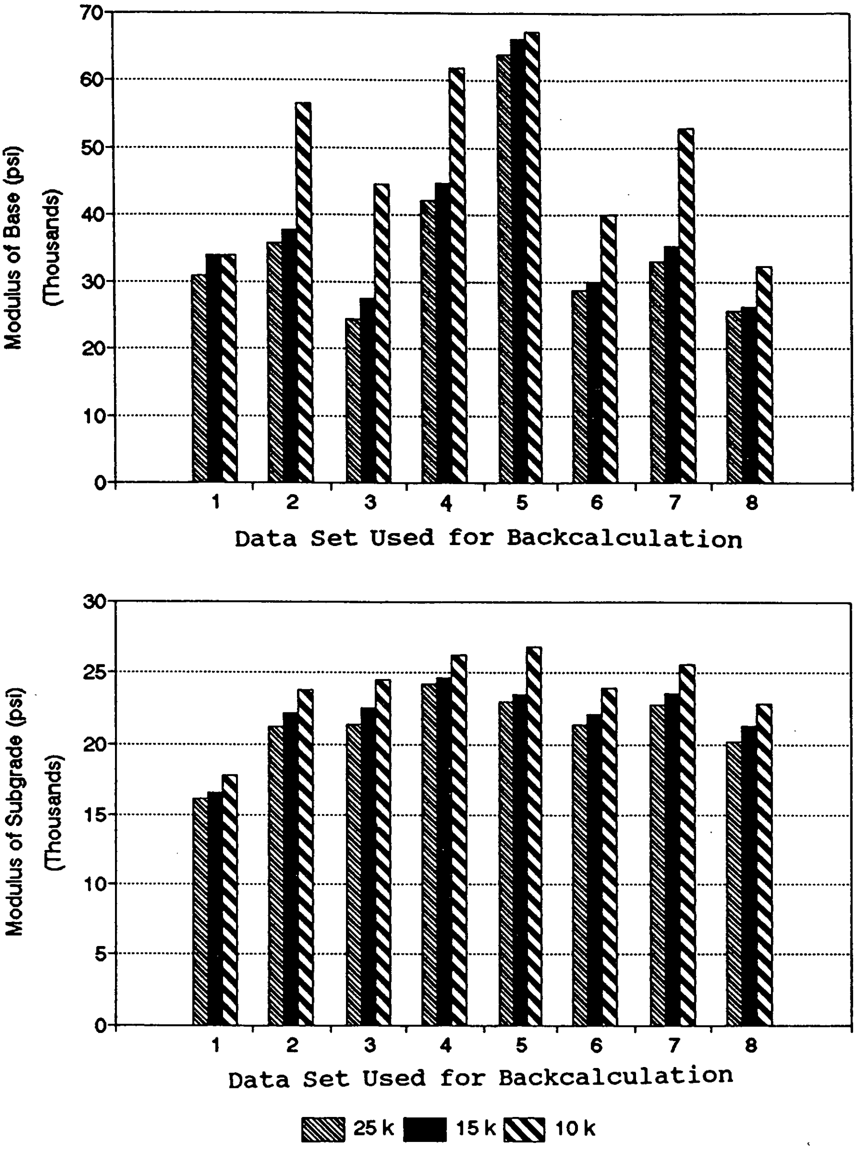


FIGURE 4 Comparison of layer moduli of base (*top*) and subgrade (*bottom*) backcalculated from various data sets imparted by one FWD.

of known data points relative to the number of parameters to be determined), the more statistically appropriate the outcome will be. This matter was investigated.

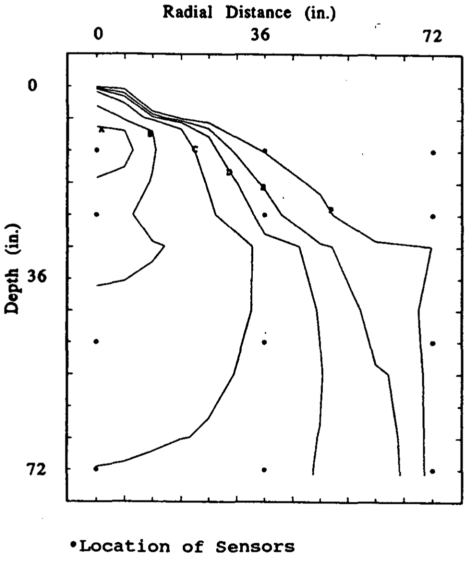
In Figure 4, three other data sets are presented. Moduli reported for Data Set 6 correspond to moduli backcalculated utilizing a total of 14 or 15 deflections. For Data Set 7, data from the embedded receivers were used. Twelve deflection points were available (see Figure 3). For the last series, Data Set 8, deflection from all surface sensors plus the 12 deflection points used in Data Set 7 were used (i.e., a total of 16 to 19 deflections).

To backcalculate moduli from these data sets, the pro gram BISDEF had to be slightly modified. Generally, the deflection basin fitting for Data Sets 6 and 8 did not yield satisfactory results (average absolute differences of greater than 10 percent). However, deflections from Data Set 7 could be matched relatively close (average absolute differences of about 5 percent). Once again, as the deflections from the nonlinear regions were minimized, the quality of basin fitting improved.

Because of an unsatisfactory deflection basin-fitting process, it would be difficult to draw definite conclusions. However, using deflections from Data Set 6 generally yields base moduli that are approximately equal to or slightly less than those obtained from surface sensors. However, the subgrade moduli are always greater from Data Set 6 compared with those backcalculated from surface sensors.

Typically, moduli obtained for the base and subgrade employing Data Set 7 are greater than those obtained from Data Set 1 (surface deflection data only). The reason for this matter is that deflections for Data Set 7 are from regions that are less affected with the load-induced nonlinearity.

Base moduli from Data Set 8 are always less than those determined from the surface deflections. By way of contrast,



Description of Contours (Numbers in percent strain)

Load, kip	A	B	C	D	E	F
25	0.05	0.03	0.01	0.005	0.003	0.001
20	0.04	0.024	0.008	0.004	0.0024	0.0008
15	0.03	0.018	0.006	0.003	0.0018	0.0006
10	0.02	0.012	0.004	0.002	0.0012	0.0004
5	0.01	0.006	0.002	0.001	0.0006	0.0002

FIGURE 5 Strain distribution in pavement section used in this study caused by loads applied to a 6-in.-radius plate.

TABLE 2 Degree of Load-Induced Nonlinearity at Sensor Locations for Load Plate with 6-in. Radius

Data Set	Sensor	Load Level, kips			
		5	10	15	25
2	11	N	N	N	N
	12	L	L	L	L
	13	L	L	L	L
3	21	Q	N	N	N
	22	L	L	L	Q
	23	L	L	L	L
4	31	Q	Q	Q	N
	32	L	Q	Q	Q
	33	L	L	L	L
5	41	L	Q	Q	Q
	42	L	L	Q	Q
	43	L	L	L	L

N: Nonlinear
Q: Quasi-Linear
L: Linear

the subgrade moduli are always greater than those obtained from surface deflections.

COMPARISON OF DEFLECTIONS

A good indication of the quality of the backcalculation techniques is how well the measured deflections and the deflections determined from the backcalculated moduli compare. If the two compare closely, one can conclude that the existing processes are adequate.

Another motivation for comparing the measured and calculated deflections is to verify a widely implemented assumption that accurate design parameters can be obtained from elastostatic backcalculation programs provided that heavy loads (corresponding to expected traffic loads) are applied to the pavement. In other words, if one imparts large loads to the pavement surface and measures deflections affected with load-induced nonlinearity; and if one backcalculates moduli from an algorithm (such as BISDEF), which is based on the assumption that materials behave linear elastically; and finally, if the backcalculated moduli are used to determine the stresses, strains, and deformations using an elastostatic algorithm (similar to BISAR), the measured and calculated stresses, strains, and deformations at that load level will be similar. This matter is investigated next.

The comparison of measured and calculated deflections from one data set is presented in Figure 6. The calculated deflections are determined from a layer modulus backcalculated from deflection Data Set 1 (surface sensors only) caused by 25 kips of load applied by an FWD. The comparison of deflections from all devices and all load levels is included in Chai (*6*).

The deflection contour lines obtained from the deflections measured with one FWD at a load level of 25 kips is shown in Figure 6*a*. The contour lines are rather coarse because of the scarcity of deflection data. The points where data were collected are marked with a small solid point. Next to each data point, the measured deflections in mils are reflected. The contour lines are depicted as solid lines, and the contour value is reflected in bold numbers next to each line.

At a depth of about 27 in., a sharp change in the slope of the contour lines can be observed. This depth corresponds to the thickness of the AC and base layers. Sharp changes in the slopes at this level are expected because of the differences between the modulus of the base and subgrade layers. Had the data been collected with more resolution (i.e., utilizing more geophone units) a similar change in slope would have been observed at the interface of the asphalt and the base layers.

To compare the theoretical and experimental results, contour lines were again determined. However, instead of using measured deflections, deflections calculated from BISAR using backcalculated moduli were used. The contour lines for this case are presented in Figure 6*b*. Basically, the two deflection contours vary significantly. This variation is summarized and quantified in the next paragraph.

Shown in Figure 7*a* is the average percentage difference between the calculated and measured deflections for each device and each load level for Data Set 1 (surface deflections only). To obtain this value for a given device and load level, the corresponding absolute difference between the measured and calculated deflections from 15 sensor locations (12 embedded and 3 on the surface) were averaged. A small value will indicate overall agreement between the measured and calculated value.

In Figure 7*b*, the average absolute difference from deflection-basin fitting for each device and load level is shown. The two parameters (average percent difference in deflections and av-

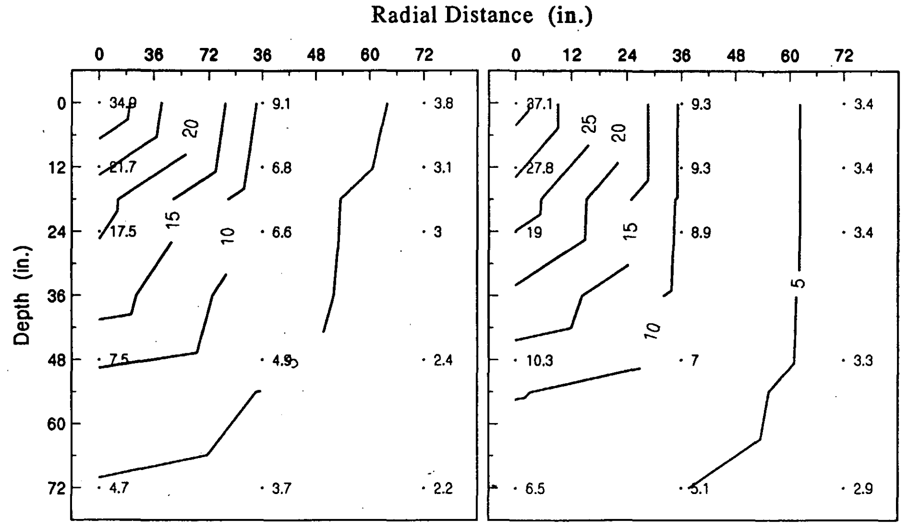


FIGURE 6 Comparison of measured (*left*) and calculated (*right*) deflections from one FWD.

Device	1	2	3	4	5	6	7
Equipment	Dynatest	Kuab	WES 16-kip	Road Rater	Dynaflect	HWD	Phoenix

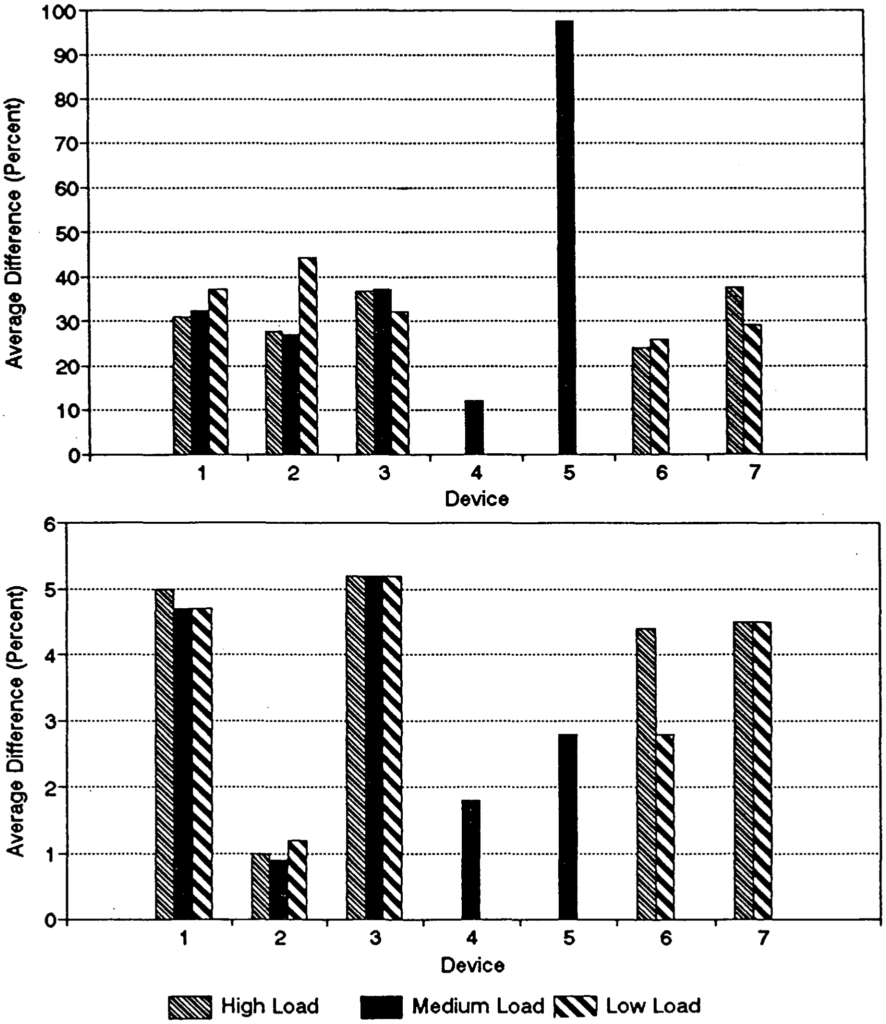


FIGURE 7 Comparison of measured deflections with deflections calculated from backcalculated moduli using Data Set 1: *top*, measured versus calculated; *bottom*, from basin fitting.

erage absolute difference in the basin fitting) go hand in hand. It is not appropriate to compare experimental deflections with theoretical deflections obtained from backcalculated moduli where the basin fitting is not done closely.

For the surface deflections, the basin fitting is done reasonably well for every case. The minimum and maximum average absolute differences in basin fitting are 0.9 percent and 5.7 percent, respectively (Figure 7*b*). However, the theoretical and experimental deflections within the body of pavement do not compare well at all. The average percent differences from Figure 7*a* vary from a minimum of about 24 percent to a maximum of about 98 percent.

Results from other data sets are not shown here, for the sake of brevity. For Data Sets 2 through 4 as deeper data sets were used, the difference among various devices and load levels were less pronounced. This may suggest that the discrepancy between the measured and calculated data is caused by the inherent problems in the data reduction instead of the type and magnitude of load. The agreements between the measured and theoretical deflections for Data Set 5 are much better than those obtained from surface deflections as reflected in the figure. The average difference is always less than 20 percent.

The last three data sets (Data Sets 6 through 8) should logically yield relatively good agreement between the measured and calculated deflections. Basically, data from all data points were used in deflection basin fitting. For Data Set 6, the average absolute differences are less than 20 percent. Such a relatively good agreement may be fortuitous because of large absolute differences found from deflection basin fitting. Similar results were obtained from Data Sets 7 and 8.

DETERMINATION OF STRESSES

The modulus of each layer backcalculated from various deflection data sets can be used to calculate stresses at interface directly under the load from progam BISAR. The normalized radial tensile stresses and normalized vertical stresses at Interface 1 (AC and base layers) and Interface 2 (base and subgrade layers) are computed. To obtain normalized stresses, the stress calculated from BISAR at each interface was divided by the loading stress applied at the surface. These data can be used to estimate the amount of stress generated in the pavement system by each device.

Vertical Stresses

Vertical stresses determined from various data sets are reported in Chai (*6*). For one FWD, the calculated vertical stresses are shown in Figure 8. For Interface 1, the highest vertical stresses were obtained when backcalculated moduli from Data Set 1 (surface sensors) or Data Set 5 (deepest three sensors) were used. For Interface 2, the stresses are more or less independent of the data set used or the level of load applied.

Stresses in a pavement section are directly related to the Young's moduli of different layers. This matter is well reflected in Figure 8. Considering the fact that the modulus of the AC layer was assumed to be constant for Data Sets 2 through 5, the vertical stress at Interface 1 increases as the modulus of the base layer increases (see Figure 4). Similarly, the stress at Interface 2 decreases with an increase in the modulus of subgrade.

For the other three falling weight devices, similar trends occur. For these three devices, the highest vertical stress at Interface 1 occurs when Data Set 5 was used. Data Set 5 corresponds to the three deflections least affected by the load-induced nonlinearity. This trend is significantly important from a design point of view. Vertical stresses calculated using the conventional surface sensors are in the order of 25 to 100 percent lower than those determined from Data Set 5. It is premature to make any conclusions on the basis of one site. However, this may suggest that (against popular belief), for the study of rutting, the use of moduli obtained from techniques using lower load intensities may be desirable. More study of this phenomenon is highly recommended.

On the other hand, the stresses obtained at Interface 2 are usually most critical from surface deflections. However, for most data sets (except Data Set 5), the vertical stress at Interface 2 is reasonably independent of the data set used in backcalculation or the load level applied to the pavement.

For WES 16-kips vibrator and the Road Rater, the trends in variation in vertical stresses at the two interfaces as a function of data set used follow trends that are similar to those of the falling weight devices described above.

The results from the Dynaflect device indicate that the stresses obtained from Data Sets 1 through 4 are similar and independent of data set used. One possible explanation for this closeness is that deflections from all four data sets correspond to the linear range and therefore yield similar moduli and stresses. Deflections from Data Set 5 for Dynaflect suffer from a low signal-to-noise ratio and therefore are not as reliable.

In general, the maximum values for the first interface vary between about 20 and 42 lb/in.^{2}, and for the second interface between 2.5 and 3.6 lb/in.^{2}. The minimum vertical stresses varied from 10 to 18 lb/in.^{2} for Interface 1, and from 1 to 3.4 lb/in.^{2} for Interface 2. This figure may be an indication of the device dependency and load dependency of the results obtained from the deflection-based NDT devices.

Radial Stresses

Radial stresses determined from various data sets from one FWD are presented in Figure 9. Basically, stresses from all devices (except the Dynaflect) exhibit similar behaviors. For Interface 1, Data Set 5 yields the smallest radial tensile stresses. Simultaneously, radial stresses at Interface 2 affected by the same set of data (Data Set 5) result in the largest tensile stresses. Practically speaking, moduli obtained from deflection data less affected by the load-induced nonlinearity may result in higher radial stresses at the base-subgrade interface.

For Interface 1, the variability in the results is large for Data Set 1 because of a large variation in the modulus of AC layer. As for Data Sets 2 through 5, a constant modulus value was assigned to the AC layer, and the stresses follow a predictable pattern. Basically, moduli from Data Set 3 are the most critical values in terms of stresses at Interface 1.

As for the vertical stresses, the signal-to-noise ratio for the data collected with the Dynaflect is small, and this matter may have caused the amount of scatter seen in the data.

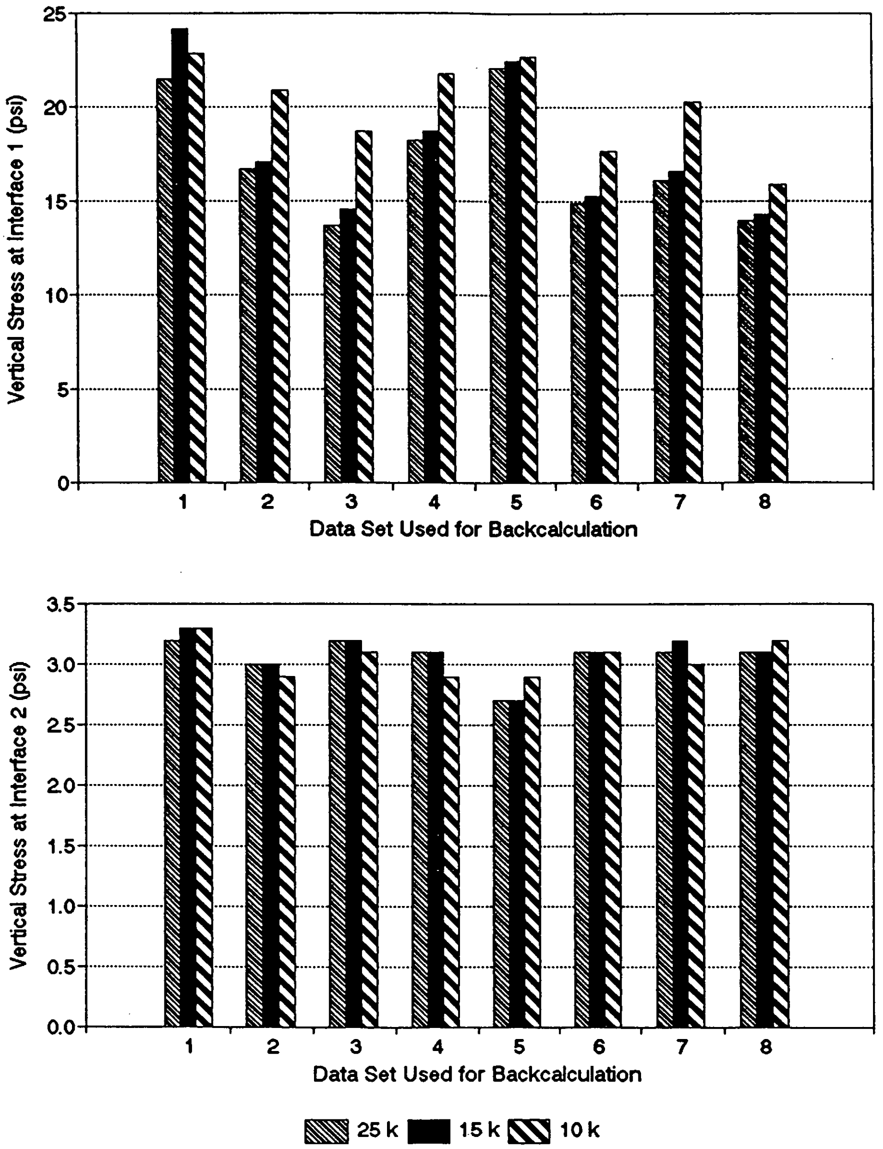


FIGURE 8 Variation of vertical stresses at two interfaces for various data sets caused by a 9,000-lb load applied using moduli backcalculated from Dynatest FWD: *top*, Interface 1; *bottom*, Interface 2.

Large variations in the minimum and maximum values of tensile radial stresses at both interfaces as a function of the type of device used and the nature of load imparted were evident.

SUMMARY AND CONCLUSIONS

Backcalculation methods for determining elastic moduli from deflection profiles obtained by nondestructive testing devices have provided researchers with a tool to improve pavement design. Nevertheless, it is important to verify the accuracy of backcalculating methods. The use of a relatively low-cost geophone system to achieve this goal is described in this paper.

The process of installation of the geophone system is presented. An explanation of the approach used to determine deflections using geophones is reviewed. Deflections from seven NDT devices were used to determine the closeness between the measured and calculated responses of a pavement section. The limitations of one backcalculating algorithm as a function of amplitude of loads imparted and the type of device used were examined. The capability of modeling the behaviors of pavements from backcalculated moduli using elastostatic programs was investigated. The stresses critical to design were also examined.

Because only one site was used, it may be premature to make any conclusions. However, the results from this study reveal some items that may be of interest for further theoretical and experimental investigations. These items are

1. Deflection basin fitting would not yield a close match between the theoretical and measured deflections when some

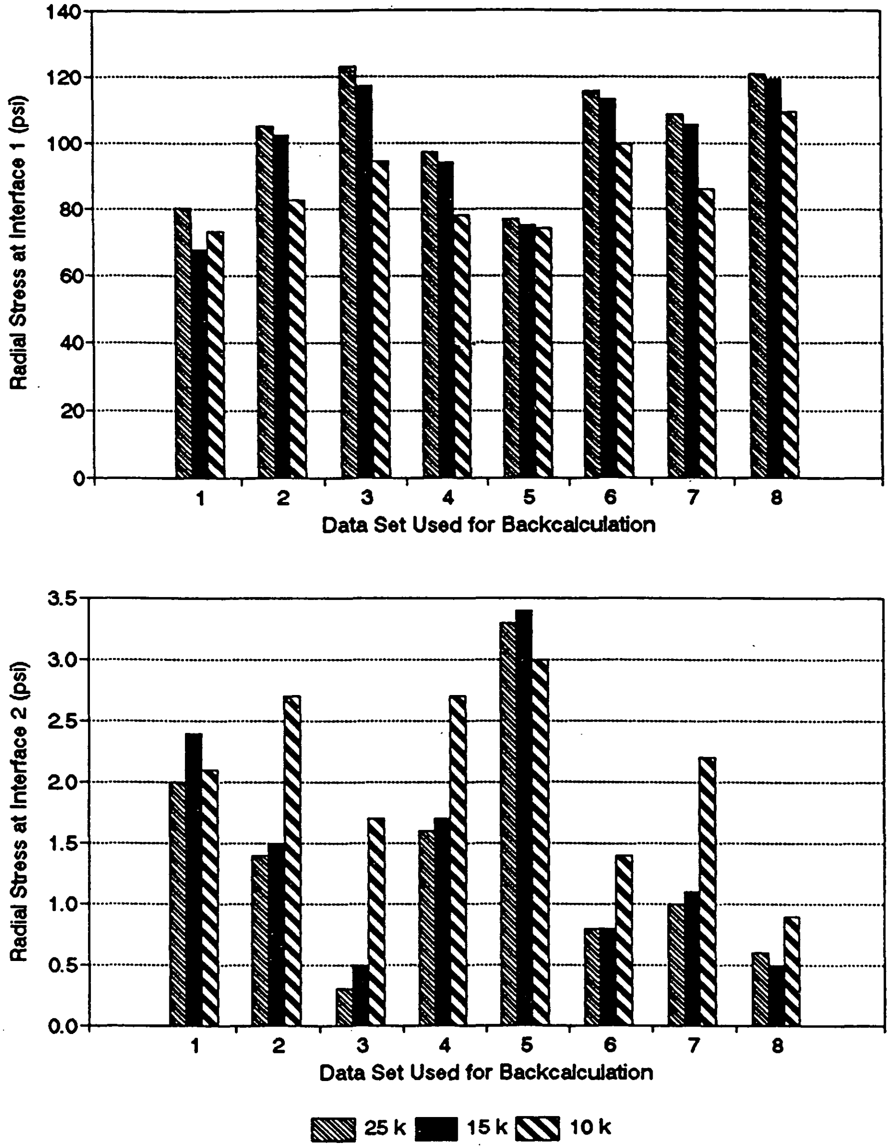


FIGURE 9 **Variation in radial stresses at two interfaces for various data sets caused by a 9,000-lb load using moduli backcalculated from one FWD: *top*, Interface 1; *bottom*, Interface 2.**

of the deflections are obtained from the regions contaminated with load-induced nonlinearity; conversely, the deflection basin fitting may yield a close match when none of the deflections are measured in the nonlinear region;

2. Theoretical deflections within the body of a pavement from backcalculated NDT moduli (using surface deflections), may not be representative of deflections measured at similar points;
3. Theoretical deflections, calculated from measured deflections away from the load (not affected with load-induced nonlinearity), may be close to measured deflections at similar points;
4. Vertical compressive and radial tensile stresses determined from moduli backcalculated from conventional surface deflections may be conservative.

Finally, many of the conclusions and statistics made here are further analyzed and substantiated in Chai (*6*). Because of space limitations, only the major steps are highlighted herein. The interested readers are referred to that publication for a comprehensive discussion.

ACKNOWLEDGMENTS

This work was supported by the University of Texas at El Paso. The instrumentation of the site and the collection of the raw data were done with the technical and monetary assistance of the Army Corps of Engineers Waterways Experiment Station. Al Bush, Don Alexander, and Ross Bensten of that organization are acknowledged for their technical assistance, hard work, and friendship. Dennis Hiltunen of Penn State was of great help in collecting the field data.

REFERENCES

1. T. Scullion, R. C. Briggs, and R. L. Lytton. *Using the Multidepth Deflectometer to Verify Modulus Backcalculation Procedures*. ASTM Special Technical Publication 1026, Philadelphia, Pa., 1989, pp. 90–101.
2. V. Tandon and S. Nazarian. *Comprehensive Evaluation of Sensors Used for Pavement Monitoring*. Research Report 913-1F. University of Texas at El Paso, 1991.
3. R. A. Bensten, S. Nazarian, and J. A. Harrison. *Reliability Testing of Seven Nondestructive Pavement Testing Devices*. ASTM Special Technical Publication 1026, Philadelphia, Pa., 1989, pp. 278–290.
4. W. R. Hudson, G. E. Elkins, W. Uddin, and K. T. Reilley. *Pavement Condition Monitoring Methods and Equipment, Phase I—Part 1 Evaluation of Deflection Measuring Equipment*. Report FH67/1. ARE Inc., Austin, Tex., 1986.
5. S. Nazarian, E. Y. Chai, and D. R. Alexander. Pavement Instrumentation for Verifying Elastic Theory. *Proc., 2nd International Symposium on State-of-the-Art of Pavement Response Monitoring Systems for Roads and Airfields*, West Lebanon, N.H., Sept. 1991.
6. Y. E. Chai. *In Situ Verification of Layer Theory for Pavement Design and Rehabilitation*. Master's thesis. University of Texas at El Paso, 1990.