increases in surface-course thickness or in the distance from the load center decrease the α-value. The β- and γ-parameters do not seem to be affected appreciably by the above factors.

4. It is believed from the favorable agreement between calculated and measured responses that transfer-function theory appears to be capable of predicting static- or repeated-load deflections of flexible pavements.

ACKNOWLEDGMENT

We extend our sincere thanks to the authorities of the Joint Highway Research Project of Purdue University for financial support of this research. The opinions and conclusions expressed are ours and do not necessarily reflect those of the sponsoring agency.

REFERENCES

1. C.B. Drennon and W.J. Kenis. Response of a Flexible Pavement to Repetitive and Static Loads. HRB, Highway Research Record 337, 1970, pp. 40-54.
2. T.M. Eggleston and C.W. Mathews. Application of Several Methods for Determining Transfer Functions and Frequency Response of Aircraft from Flight Data. National Advisory Committee for Aeronautics, TR-Rept. 1204, 1954, pp. 1-24. NTIS: NACA-TR-1204.
3. P.A. Crafton. Shock and Vibration in Linear Systems. Harper, New York, 1961.
4. J.H. Goldberg. Automatic Controls: Principles of Systems Dynamics. Allyn and Bacon, Boston, 1964.
5. T.K. Puchalka and A. Wozniak. Elements and Circuits for Automatic Control. Boston Technical Publishers, Cambridge, MA, 1968.
6. S.A. Swami, W.H. Goetz, and M.E. Harr. Time and Load Independent Properties of Bituminous Mixtures. HRB, Highway Research Record 313, 1970, pp. 63-78.
7. R.E. Boyer and M.E. Harr. Predicting Pavement Performance. Transportation Engineering Journal of ASCE, Vol. 100, No. TE2, May 1974, pp. 431-442.
8. R.E. Boyer. Predicting Pavement Performance Using Time-Dependent Transfer Functions. Purdue Univ. and Indiana State Highway Commission, Lafayette, IN, Joint Highway Research Proj. 32, Sept. 1972.
9. G.A. Ali. A Laboratory Investigation of the Application of Transfer Functions to Flexible Pavements. Purdue Univ., Lafayette, IN, Ph.D. thesis, Aug. 1972.
10. W. Heukelom. Analysis of Dynamic Deflections of Soils and Pavements. Geotechnique, Vol. 11, No. 3, Sept. 1961, pp. 224-243.
11. N.M. Isada. Detecting Variations in Load-Carrying Capacity of Flexible Pavements. NCHRP, Rept. 21, 1966.
12. N.M. Isada. Impulsive Load Stiffness of Flexible Pavements. Journal of the Soil Mechanics and Foundations Division of ASCE, Vol. 96, No. SM 2, March 1970, pp. 639-648.
13. G.Y. Baladi and M.E. Harr. Nondestructive Pavement Evaluation: The Deflection Beam. TRB, Transportation Research Record 666, 1978, pp. 19-26.

Publication of this paper sponsored by Committee on Flexible Pavement Design.

Evaluation of Pavement in Florida by Using the Falling-Weight Deflectometer

JATINDER SHARMA AND R. N. STUBSTAD

A method is presented by which mechanical properties of a pavement system can be determined by using nondestructive test methods that are now available. The ultimate goal is the establishment of rehabilitation criteria for existing flexible pavements that use purely analytical (as opposed to empirical) relationships. More specifically, the use of the falling-weight deflectometer (FWD) is discussed. Several sections of Interstate 75 in Florida were chosen in order to determine material characteristics of the pavement layers. These sections were also tested with the Dynaflect apparatus. Data developed from the FWD and Dynaflect deflections were accumulated and elastic moduli for the typical section were determined by using a computer program developed at the Florida Department of Transportation: in situ stress-dependent elastic moduli, four layers (ISSEM4). The elastic moduli were then compared with other test results, and a good correlation was indicated. How such mechanical properties may be used in an appropriate structural analysis to better locate and control distress parameters in the pavement system is outlined. Such analysis is possible from the knowledge obtained in situ of the various structural layers involved.

For many years the Florida Department of Transportation (FDOT) has used various deflection concepts to monitor both local and Interstate road networks. Generally, it has been found that deflection alone is not an adequate indicator of pavement performance or loss of serviceability. For example, many situations have been observed in which deflections remained low, even though significant load-associated pavement deterioration was visibly taking place.

Surface deflection may be interpreted as the sum of the vertical strains throughout each structural layer below. If a weakness should develop in one or more of these layers, it may not necessarily change the total deflection significantly; e.g., a relatively thin layer might contribute little change to the center measurement of a deflection-measuring device. A more-indicative measure of distress is thus necessary.

In order to further understand and evaluate pavement deterioration and ultimately recommend corrective rehabilitation and management strategies, it was decided to try to isolate problem areas in terms of which layer or layers were instrumental in the deterioration of Interstate 75 in northern Florida. Also, it was hoped that performance criteria based on derived material properties could be developed.

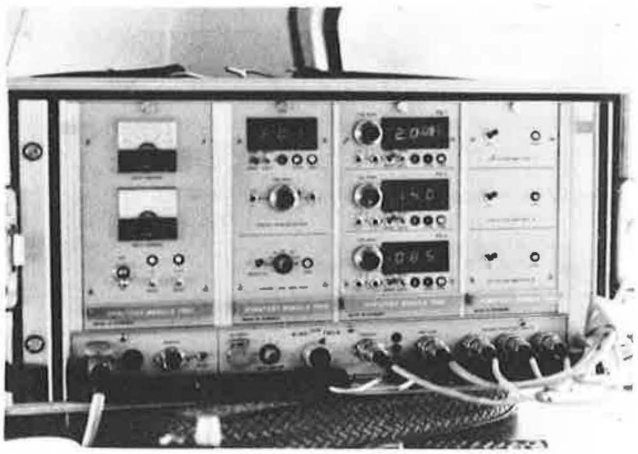
On the basis of work done in Europe, the falling-weight deflectometer (FWD) was chosen to carry out a layered-system (mechanistic) analysis of the pavement structure. Approximately 180 lane-km (110 lane-miles) of Interstate were tested and analyzed (see Figure 1).

FWD AND ASSOCIATED EQUIPMENT

Use of the FWD has been well documented elsewhere (1-3). Briefly, the basic idea behind the development of the FWD

Figure 1. Falling-weight deflectometer (FWD) on I-75 near Gainesville, Florida.

Figure 2. Dynatest 7800 registration equipment for FWD testing.



was attractive, i.e., to simulate the effect of a moving-wheel load with a nondestructive test apparatus. This is accomplished in terms of both stress and load and, to a lesser degree, in terms of duration of load. Not simulated is the rotation of principal stresses as they occur under a moving-wheel load. Although the FWD satisfactorily simulates the duration of load near the surface, when the moving-wheel load is measured at greater depths, there is a somewhat longer duration than that measured by the FWD.

Nevertheless, the possible shortcomings of the wheel-load-simulating FWD loading system have been largely dispelled by several research projects (4,5). In these projects the deflections, stresses, and strains throughout the pavement systems were compared for similar moving-wheel-load and FWD-imposed forces. The correspondence of the two was remarkably satisfactory for all three parameters (within 10 percent). Therefore, it was felt that the FWD system would be beneficial to the state of the art. Features of the FWD are (a) a capable load range of about 13-50 kN (3000-11 000 lbf), (b) an associated loading time (approximately half-sine-formed) of 26 ms, and (c) vertical deflections that may be taken at any desired position from the center of the loading plate outward along the deflection basin. The peak value of the stress level under the loading plate and the corresponding peak values of deflection are digitized and recorded on the Dynatest 7800 registration equipment (Figure 2).

On the assumption that the elastic moduli of materials may be derived from deflection tests, it was felt that the FWD would correspond better to conditions that would be appropriate for properties relevant to the wheel (axle) load than would the several available steady-state loading systems now in use in the United States, provided that an appropriate structural analysis based on surface deflections could be carried out.

MULTILAYERED REVERSE-ITERATIVE COMPUTER PROGRAM

Although approximate methods that can be performed on a calculator have been devised and documented for deriving stiffness values based on deflection measurements (6), it was felt that a quicker and more foolproof method would be the development of a reverse elastic-system computer program. Such an approach was attractive because the derived parameters, due to the similarity between FWD and moving-wheel-load effects, would be relevant for direct measurement of performance criteria of traffic-associated loadings.

Although a finite-element program, in which each element would have been modeled by using the best information available for each respective structural material, would have been best suited to derive values of stiffness of modulus, such an approach would have required too much computer time per analysis to ascertain these stiffness relationships from each set of FWD deflection data. At the same time, it should be pointed out that most road-building materials do not respond linearly; i.e., different states of stress result in different apparent stiffnesses (secant moduli) for the same material. If a linear-elastic program, in which calculated versus measured deflections are matched by juggling E-values, is used to model the pavement system, gross errors will result even if the stress-dependent nature of the materials (especially the semi-infinite subgrade) are comparatively minimal.

These factors predicted that the linear-elastic ELSYM5 program (7) could nevertheless be employed, although it would have to be modified so that variable stiffnesses that depended on the state of stress under each corresponding deflection sensor could be used for calculating the total deflection. The individual solution is valid for that particular deflection position only. A separate calculation is necessary for each measured deflection along the deflection basin.

By using principles of the method of equivalent thicknesses as well as Boussinesq's equations, the iteration procedure was streamlined and implemented so that a unique solution could be quickly obtained. The program is known as in situ stress-dependent elastic moduli, four layers (ISSEM4), documented elsewhere (8).

The only additional input quantities needed, other than those normally used in the ELSYM5 input format, are the measured deflections at the design or FWD load, the load magnitude and radius, and the K_2-values for the second (base), third (subbase), and fourth (subgrade) layers, where the stiffness or resilient modulus (E) assumes the form

$$E(\text{or } E_0) = K_1 \sigma_x^{K_2} \quad (1)$$

where

σ_x = some selected dynamic stress parameter in the modeled material,

E = element (cylindrical column the height of the structural layer) modulus,

E_0 = surface (i.e., subgrade surface) or composite modulus, and

K_1 and K_2 = material characterization constants.

The K_2-values can be obtained through FWD tests that are conducted at different stress levels.

Finally, the iterative ISSEM4 program is seeded with a set of E-values to start the iteration process. The results are output in the form of derived variable stiffnesses for the three unbound structural layers and an E_1-value for the centerline; all of these are characterized by the corresponding measured versus derived deflections at several distances r from the load (up to seven positions). The derived K_1-values for each structural material (layer) are also listed in the output.

The most useful and interesting aspect of the results of these calculations is, of course, the centerline stiffnesses, which can be used in the calculation of critical stresses and strains for a given design-load configuration. If the design load is significantly different from the imposed FWD load level used in the analysis, the centerline E-values may be altered according to the Equation 1 relationship(s) mentioned above.

BASIC APPROACH TO ISSEM4 PROGRAM

As the ISSEM4 program now exists, it is necessary to assign certain stiffness model parameters (K_2-values) for layers 2, 3, and 4 in the pavement structure. Again, a model of the form specified by Equation 1 was chosen. This model may be used to assist in the optimal use of the ISSEM4 program as follows.

Generally speaking, the farther a deflection reading is from the load, the deeper the materials in the structure affect that deflection. In fact, as Ullidtz (6) has shown, if the equivalent thickness (h_e) of the pavement structure above the subgrade is defined as

$$h_{e,m} = 0.85\left[h_1 \sqrt[3]{E_1/E_m} + h_2 \sqrt[3]{E_2/E_m} + \ldots h_{m-1} \sqrt[3]{(E_{m-1})/E_m}\right] \quad (2)$$

which is equal to or less than the distance r from the center of the FWD load to a deflection sensor, the surface modulus (E_0) of the subgrade will be very close to

$$E_{0,m} = [\sigma_0 a^2 (1 - \nu^2)]/(rS_{0,r}) \quad (3)$$

where

σ_0 = the loading-plate stress level,
a = radius of the plate,
ν = Poisson's ratio, and
$S_{0,r}$ = deflection reading at position r.

Since the falling weight may be used over a wide range of stress levels, the K_2-value associated with the subgrade may now be easily calculated from FWD test results alone by calculating the $E_{0,m}$-values for a series of stress levels σ_0.

Since σ_x in Equation 1 is approximately proportional to σ_0, a regression analysis (in a log-log form) between the variables σ_0 and $E_{0,m}$ will yield the subgrade slope K_2.

By selecting proper deflection sensor positions, the K_2-values associated with other areas of the pavement structure can be deduced, and this process generally results in a better-than-educated-guess estimate of the three K_2-values needed for the ISSEM4 input format. Furthermore, since the effective layer stiffnesses that correspond to the various distances r from the actual load associated with the FWD or other deflection-testing devices vary significantly, even for very small values of K_2, an educated guess at these values will produce far more reliable centerline stiffness results than if no stress sensitivity were considered. This illustrates a fundamental error normally inherent in most nondestructive-testing analysis techniques, namely, that the layered-system E-values are typically modeled as constant in a horizontal direction. This assumption, especially with regard to the subgrade, will result in gross errors when the deflection basin is used to derive centerline stiffness.

SAMPLE RUNS OF ISSEM4

The structural dimensions and materials of one of the areas of I-75 that was investigated are as follows (1 mm = 0.039 in):

Material	Thickness (mm)
Asphalt concrete, h_1	178
Limerock base, h_2	265
Subbase, h_3	310
Subgrade, h_4	∞

FWD test results at a typical point in the inner-wheel path of the traffic lane gave the set of measured deflections shown below for a plate diameter of 300 mm (11.7 in) (1 mm = 0.039 in; 1 kPa = 0.145 lbf/in²):

Distance from Load, r (mm)	Deflection, $S_{0,r}$ σ_0 = 695 kPa	(μm) σ_0 = 354 kPa
0	205	93
300	150	66
450	118	51
750	77	32
1200	41	18

By using Equation 3, the values of $E_{0,m}$ that correspond to the subgrade at two stress levels may be calculated with the deflections at r = 1200 mm (47 in). For σ_0-values of 695 and 354 kPa (100.8 and 51.3 lbf/in²), $E_{0,m}$ = 279 and 324 MPa (40 455 and 46 980 lbf/in²), respectively. By using the model depicted in Equation 1, the K_2-exponent of σ_0 versus $E_{0,m}$ then becomes K_2 = -0.22. This value is rounded to -0.20.

If we keep in mind that the determination of a K_2-value that corresponds to a distance $r \sim h_e$ above layers 2 or 3 will produce a composite slope, which includes the materials below the layer in question, the K_2-values may nevertheless be estimated by considering the superposition nature of the E_0-values thus obtained. Initially, this is carried out in a qualitative manner, but finally the whole deflection basin can be compared with the final calculated deflection values to obtain the best solution.

In this case, the final K_2-values and the estimates of Poisson's ratio were

Layer	ν	K_2
1	0.35	-
2	0.25	-0.20
3	0.35	-0.40
4	0.35	-0.20

Figure 3 shows how the layered-system E-values vary according to the $E = K_1\sigma_x^{K_2}$ relationship for all r's ≤ h_e above the layer in question. The results are quite reasonable; the average asphalt temperature was about 18°C (64°F) (tested in January 1979). Interestingly, the same series of FWD tests was run about seven months later, during the summer of 1979. The average asphalt temperature was then about 27°C (80°F), an increase of 9°C (16°F).

By using the method outlined above, the K_2-values were fixed, and the resulting ISSEM4 output is shown in Figure 4.

It is significant to note that even though the asphalt modulus E_1 was lower during the summer, as expected, the center FWD deflection actually decreased, which indicates a stiffer overall surface modulus (composite stiffness). A comparison of the two ISSEM4 outputs reveals why: The stiffnesses of the unbound materials increased during the summer, whereas the stiffness of the asphalt concrete (AC) decreased. The overall effect at this test point and most others on I-75 was a lower deflection; this is quite the opposite of what one would expect by regarding changes in asphalt stiffness due to temperature variation alone. This was the general tendency for all sections that were investigated along I-75.

Thus, it was possible to see clearly what was happening seasonally on this Florida section of Interstate roadway, which enabled a more rational use of critical stresses and strains relative to performance criteria through what is now

Figure 3. ISSEM4 outputs that depict derived stiffnesses for a four-layer system from FWD tests: winter analysis.

ELASTIC SYSTEM 2 - I-75 ALACHUA COUNTY SBT STA 3A, WINTER FWD ANALYSIS

	R1	R2	R3	R4	R5	R8
MEASURED DEFLECTIONS: MICRO-METERS	205.00000	150.00000	118.00000	(not used)	77.00000	41.00000
CALCULATED DEFLECTIONS: MICRO-METERS	206.25656	150.06708	121.14775	96.80402	77.14532	40.96275
MEASURED DEFLECTIONS: MILS	8.07085	5.90550	4.64566	0.0	3.03149	1.61417
CALCULATED DEFLECTIONS: MILS	8.12032	5.90814	4.76959	3.81117	3.03721	1.61270
SIG12, MAX PRIN, CENTER LAYER 2, MPA	-0.06465	-0.04620	-0.03283	-0.02300	-0.01604	-0.00543
SIG12, MAX PRIN, CENTER LAYER 2, PSI	-9.37737	-6.70052	-4.76113	-3.33518	-2.32585	-0.78771
SIG13, MAX PRIN, CENTER LAYER 3, MPA	-0.02559	-0.02235	-0.01900	-0.01561	-0.01253	-0.00602
SIG13, MAX PRIN, CENTER LAYER 3, PSI	-3.71122	-3.24114	-2.75626	-2.26336	-1.81660	-0.87314
SIGZZ4, VERTICAL, TOP OF SUBGRADE MPA	-0.01855	-0.01581	-0.01297	-0.01007	-0.00749	-0.00264
SIGZZ4, VERTICAL, TOP OF SUBGRADE PSI	-2.69113	-2.29343	-1.88047	-1.45987	-1.08562	-0.38249
E1, MPA	7080.953	6573.000	6573.000	6573.000	6573.000	6573.000
E1, PSI	1027005.063	953332.688	953332.688	953332.688	953332.688	953332.688
E2, MPA	466.953	505.624	504.000	504.000	504.000	504.000
E2, PSI	67725.688	73334.438	73098.938	73098.938	73098.938	73098.938
E3, MPA	203.698	215.804	230.120	249.089	273.972	351.000
E3, PSI	29543.840	31299.742	33376.059	36127.344	39736.254	50908.227
E4, MPA	195.188	201.737	209.933	221.027	234.335	284.363
E4, PSI	28309.563	29259.527	30448.125	32057.297	33987.344	41243.391
K1	0.0	273.37109	0.0	0.0	47.51357	88.24374
K2	0.0	-0.20000	0.0	0.0	-0.40000	-0.20000

Figure 4. ISSEM4 outputs that depict derived stiffnesses for a four-layer system from FWD tests: summer analysis.

ELASTIC SYSTEM 2 - I-75 ALACHUA COUNTY SBT STA 3A, SUMMER FWD ANALYSIS

	R1	R2	R3	R4	R5	R8
MEASURED DEFLECTIONS: MICRO-METERS	179.00000	106.00000	80.00000	(not used)	51.50000	30.00000
CALCULATED DEFLECTIONS: MICRO-METERS	178.13080	106.07993	83.79709	64.86893	51.30841	29.98637
MEASURED DEFLECTIONS: MILS	7.04723	4.17322	3.14960	0.0	2.02755	1.18110
CALCULATED DEFLECTIONS: MILS	7.01301	4.17637	3.29909	2.55389	2.02001	1.18056
SIG12, MAX PRIN, CENTER LAYER 2, MPA	-0.09798	-0.05942	-0.03792	-0.02420	-0.01546	-0.00462
SIG12, MAX PRIN, CENTER LAYER 2, PSI	-14.21110	-8.61764	-5.49914	-3.51018	-2.24263	-0.67034
SIG13, MAX PRIN, CENTER LAYER 3, MPA	-0.03472	-0.02778	-0.02315	-0.01813	-0.01388	-0.00601
SIG13, MAX PRIN, CENTER LAYER 3, PSI	-5.03517	-4.02954	-3.35733	-2.62971	-2.01253	-0.87237
SIGZZ4, VERTICAL, TOP OF SUBGRADE MPA	-0.02399	-0.01871	-0.01487	-0.01085	-0.00757	-0.00243
SIGZZ4, VERTICAL, TOP OF SUBGRADE PSI	-3.48009	-2.71427	-2.15716	-1.57306	-1.09823	-0.35287
E1, MPA	4043.707	4414.000	4414.000	4414.000	4414.000	4414.000
E1, PSI	586489.813	640196.313	640196.313	640196.313	640196.313	640196.313
E2, MPA	627.082	692.286	601.000	601.000	601.000	601.000
E2, PSI	90950.438	100407.563	87167.625	87167.625	87167.625	87167.625
E3, MPS	342.268	365.723	396.537	437.514	489.461	518.000
E3, PSI	49641.625	53043.602	57512.859	63456.004	70990.188	75129.500
E4, MPS	316.127	322.367	330.644	341.415	353.473	389.366
E4, PSI	45850.355	46755.355	47955.883	49518.012	51266.855	56472.738
K1	0.0	393.61084	0.0	0.0	88.43462	217.11929
K2	0.0	-0.20000	0.0	0.0	-0.40000	-0.10000

viewed as a relevant mechanistic analysis at the proper time, i.e., the season.

It must be emphasized here that, on the assumption that (a) elastic (linear or nonlinear) theory holds, (b) the material characterization models employed are valid, and (c) the FWD force and deflections are accurate, the solution derived through this iterative technique is unique to the accuracy allowed in the iterative process. The criterion for uniqueness is that there must be one or more deflection readings per structural layer and that these must be fairly strategically placed, approximately as outlined in the preceding discussion.

COMPARISON OF FWD WITH OTHER AVAILABLE TEST RESULTS

The temperature-modulus curve for the unaged asphalt mix that was used on the section of I-75 in the preceding sample run may be seen in Figure 5. A plot of the two E_1-values from the FWD tests shown in Figure 4 is also shown in

Figure 5. Relationship between temperature and complex modulus for the I-75 asphalt-concrete mix.

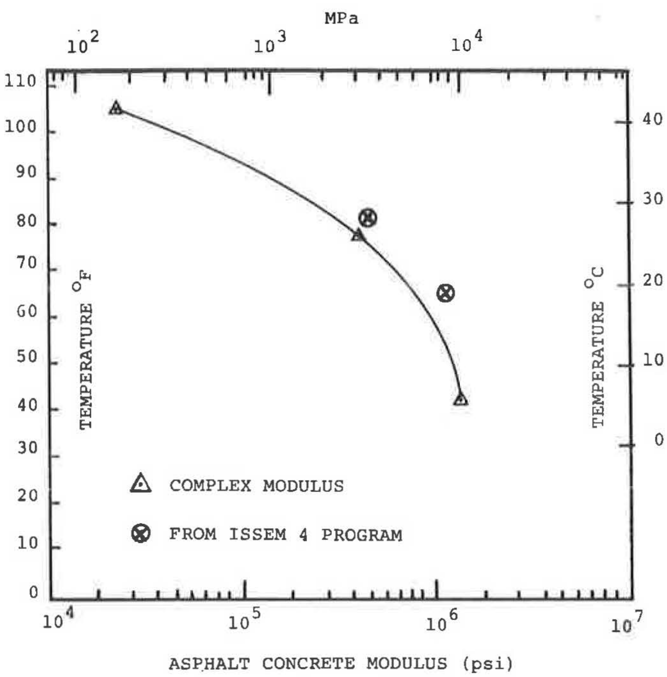


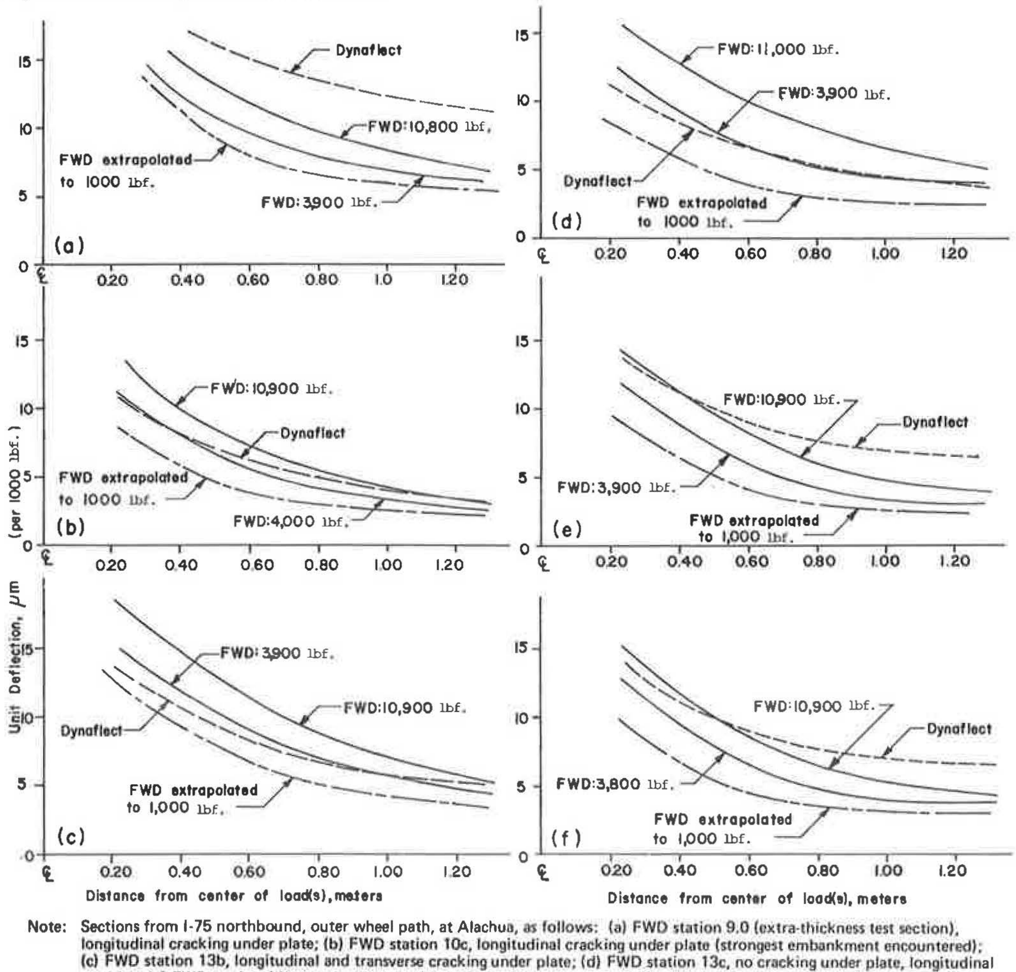
Figure 5. As expected, the mix is harder now, and the points fit the shape of the curve remarkably well.

A few static plate-bearing tests run on the trenches of I-75 subgrade compared favorably with the results derived from the FWD surface tests, although only when the change in stiffness according to Equation 1 was considered. This is in fact very important, because static plate-bearing tests are usually run at stress levels that are much higher, i.e., 10-100 times as high as those that actually occur under the completed pavement structure. The resulting difference in subgrade stiffness (depending on the magnitude of the K_2-value) may be of the order of a factor of 2 or 3.

On selected sections, the Dynaflect and FWD deflection basins were compared. Several of these sections are shown in Figure 6. The FWD deflections are normalized to units per 1000 lbf (units per 4.45 kN) to make the comparison clearer. In some cases, it may be seen that the comparison is reasonable in view of the lesser applied-load magnitude of the Dynaflect apparatus; in others, the comparison is poor.

Even though in principle the same approach may be used for evaluating Dynaflect data by using the ISSEM4 program (provided that some means of assigning K_2-values can be found), even a moderately different deflection basin will yield appreciably different stiffness values. An analysis of this sort was attempted that used the K_2-values derived from FWD tests. In accordance with the indications in Figure 5, it was found that the results often compared favorably with the FWD-derived values at a low stress level. In other cases, the derived values diverged from the FWD counterparts, most often on the high side for E_1, E_2, and E_3 and on both sides for $E_{0,4}$.

Figure 6. FWD and Dynaflect deflection basins.



Note: Sections from I-75 northbound, outer wheel path, at Alachua, as follows: (a) FWD station 9.0 (extra-thickness test section), longitudinal cracking under plate; (b) FWD station 10c, longitudinal cracking under plate (strongest embankment encountered); (c) FWD station 13b, longitudinal and transverse cracking under plate; (d) FWD station 13c, no cracking under plate, longitudinal cracking; (e) FWD station 14b, longitudinal cracking 50 mm (2 in) from plate; (f) FWD station 14b + 40 ft (12 m), longitudinal cracking under plate.

Figure 7. Stiffness values for four layers as a function of distance from FWD-ISSEM4 analysis.

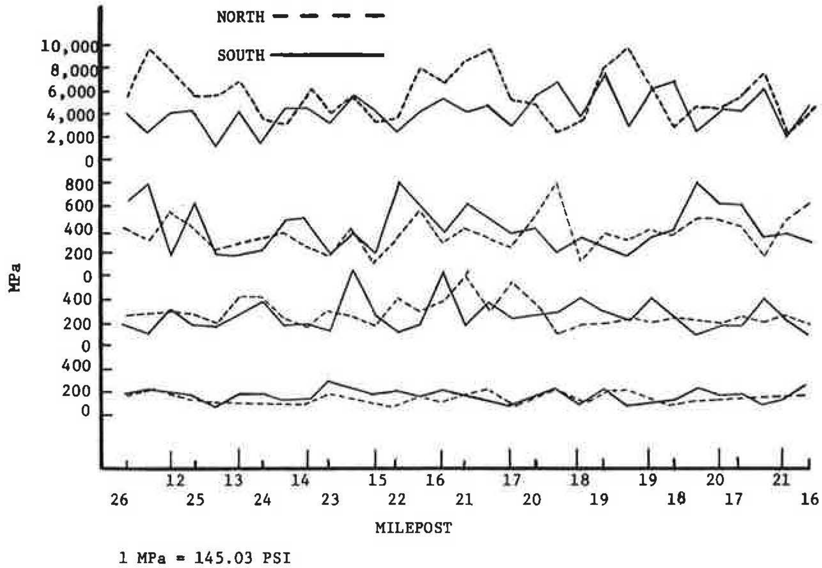


Table 1. Observed surface cracking for 24 test sections at Chiefland, Florida.

Section	Cracking (%/1000 ft^2) 50-mm AC Layer	75-mm AC Layer	Limerock Base Thickness (mm)	Subbase LBR[a]	Subgrade LBR
1B, 1A	0	0	225	32	18
10B, 10A	4	49	150	55	18
11B, 11A	8	33	225	18	18
12A, 12B	1	90	300	32	18
2A, 2B	1	0	300	55	18
3B, 3A	3	1	150	32	18
4B, 4A	6	7	300	18	18
5A, 5B	3	112	150	55	18
6B, 6A	5	118	225	32	18
7A, 7B	7	93	300	18	18
8B, 8A	139	28	150	18	18
9A, 9B	1	28	225	55	18

Note: 1 mm = 0.04 in; 1 ft^2 = 0.09 m^2.
[a]LBR = limerock bearing ratio.

At this time it is not exactly clear why this happens or which factors are causing these differences. However, stress sensitivity does not appear to be the only factor.

USE OF FWD-DERIVED STRUCTURAL PARAMETERS

The derived FWD E-values (centerline) are plotted for part of the roadway tested (Figure 7). It can be seen here that the embankment subgrade is fairly uniform; there is a subgrade surface stiffness of about 200 MPa (29 000 lbf/in^2) under the maximum FWD load of 50 kN (∿11 000 lbf) imposed at the surface of the pavement. Both directions of traffic were tested independently and plotted so that the longitudinal locations of points match up. A significant correlation between trends in $E_{0.4}$ for both directions may be shown, which indicates that this procedure is able to detect variations in E-values that are caused by moisture content or suction variations as a function of the natural landscape and geophysical conditions.

The E_1 (asphalt) values correlate well with the magnitude of cracking; low values generally had class 2 fatigue cracking, and high values had either class 1 or no cracking. In some cases, the E_1-value was high in spite of class 2 surface cracking that was associated with non-load-associated cracking. In those cases, cores revealed that the cracks appeared to extend only about 25 mm (1 in) downward from the surface.

CONCLUSIONS

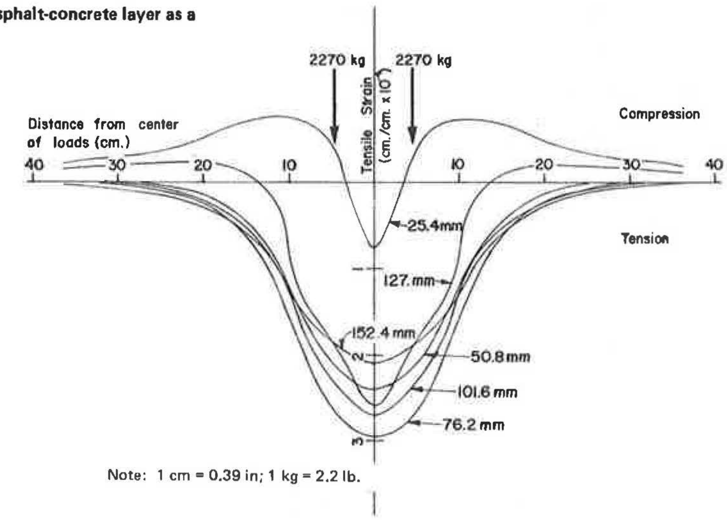
Determining in situ elastic moduli from FWD deflections by using the ISSEM4 program is of great significance and interest to the highway designer from several viewpoints:

1. The need for triaxial testing, which is required in the procedures developed by Austin Research Engineers (9) or Resource International (10), would be eliminated. For triaxial testing one has to use destructive techniques, obtain enough laboratory samples, and recompact the specimens for testing. In Florida, limerock material is widely used, and our experience and a considerable amount of testing during the past 15 years have shown that limerock gains strength with time. Plate-bearing E-values have often increased by 100 percent after construction. During the recompaction and testing for triaxial tests, we feel that the added stiffness due to cementing action is not taken into account; hence the idea of extensive destruction testing that has been suggested by other researchers becomes fruitless.

2. It is imperative that we begin to shift our design technique from purely empirically based models (AASHTO) to mechanistically based models that may help explain several distress phenomena such as the one illustrated in Table 1. In this table, the magnitude of the cracking--percentage/1000 ft^2 [percentage/90 m^2]--is shown for sections with 50 mm (2 in) and 75 mm (3 in) of asphalt concrete over 150, 225, and 300 mm (6, 9, and 12 in) of limerock base, and 300 mm of three different strengths of subbase. It is clear that in 90 percent of the cases, the 50-mm sections have cracking that is less than or equal to that of the 75-mm sections, contrary to what the AASHTO model would predict. Such phenomena can only be explained by a critical stress-and-strain type of analysis and, as can be shown for a given stiffness, the tensile strains at the bottom of asphalt concrete are higher for 75 mm than for 50 mm; hence the cracking is greater (Figure 8).

3. Once we know the proper E-values of all the layers and can attribute the variations in E-values to the known causal variables, it will help the designer to make better decisions about choice of rehabilitation alternatives, which may extend from doing nothing to resurfacing or recycling or both. The designer can then calibrate the fatigue and rut-depth criteria by knowing the history of traffic on the facility and by incorporating the stiffness values in the

Figure 8. Tensile strain at the bottom of the asphalt-concrete layer as a function of thickness.



known pavement-design models like PDMAP and VESYS. Once the model has been calibrated at that location of the roadway, the designer can calculate a thickness of overlay or recommend recycling with a greater degree of certainty.

ACKNOWLEDGMENT

This study was conducted at the Office of Materials and Research of the Florida Department of Transportation. Our special thanks go to P.E. Carpenter of the Federal Highway Administration for approving the funding of the project through preliminary engineering funds. We also wish to thank Dick Wheeler and Jerry Moxley, who carried out the entire testing program; Gary Fitzpatrick and Frank Sullivan, who helped us in reducing data, doing computer analyses, and drafting figures; and C.F. Potts and L.L. Smith for technical assistance. Dynatest Consulting, Inc., of California was retained by FDOT to furnish the falling-weight deflectometer and associated equipment, to coordinate and establish the testing program, and to initiate the ISSEM4 computer program. To this end, the Engineering Systems Design Section of FDOT furnished Fortran programming assistance in order to optimize the basic ISSEM system for wide-scale use. Warren Clary was responsible for this work, without which this project would not have been possible.

REFERENCES

1. A. A. Bohn, P. Ullidtz, R. Stubstad, and A. Sorensen. Danish Experiments with the French Falling Weight Deflectometer. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, 1972, Vol. 1, pp. 1119-1128.
2. A. Claessen, C. Valkering, and R. Ditmarsch. Pavement Evaluation with the Falling Weight Deflectometer. Proc., AAPT, Vol. 45, 1975.
3. A. Claessen and R. Ditmarsch. Pavement Evaluation and Overlay Design--The Shell Method. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.
4. P. Ullidtz. The Use of Dynamic Plate Loading Tests in Design of Overlays. Presented at Conference on Road Engineering in Asia and Australasia, Kuala Lumpur, 1973.
5. C. Busch and P. Ullidtz. Laboratory Testing of a Full-Scale Pavement. Tech. Univ. of Denmark, Lyngby, IVTB Rept. 19, 1978.
6. P. Ullidtz. Overlay and Stage-by-Stage Design. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.
7. G. Ahlborn. Elastic-Layered System with Normal Loads. Institution of Transportation and Traffic Engineering, Univ. of California, Berkeley, 1972.
8. R. Stubstad and J. Sharma. Deriving Mechanistic Properties of Pavements from Surface Deflections. Presented at International Conference on Computer Applications in Civil Engineering, Roorkee, India, 1979.
9. Austin Research Engineers, Inc. Asphalt Concrete Overlays of Flexible Pavements. Federal Highway Administration, June 1975.
10. Resource International, Inc. Development of an Overlay Design Procedure Based on Dynaflect Deflections. Federal Highway Administration, Oct. 1979.

Publication of this paper sponsored by Committee on Pavement Condition Evaluation.