A True Dynamic Method for Nondestructive Testing of Rigid Pavements with Overlays

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A new method for the dynamic evaluation of rigid pavements with overlays is presented. The method uses a vibratory nondestructive testing device (Road Rater 2008), a dynamic signal analyzer (HP3562A), and a microcomputer (MS-DOS compatible). A microcomputer FORTRAN program, GREEN-MA, has been written which uses Green's functions to calculate dynamic structural responses of layered systems subjected to harmonic loads. A backcalculation algorithm, COMDEF, has been modified to include the dynamic deflections calculated by GREEN-MA.

Dynamic theory allows the use of more data from a single test than would normally be utilized. When analyzing data from a nondestructive testing device, the typical data includes the force and peak deflection measurements. By monitoring the sensor outputs with a dynamic signal analyzer, the data set can easily be increased to include the phase shifts between sensors. Use of phase data allows the determination of a "real time" deflection basin which represents a "photograph" of the deflection basin at a given instant.

Many authors have written about the need for true dynamic analysis of nondestructive test data. The major reason for continued use of quasistatic theory in backcalculation has been the large amounts of computer time required to generate true dynamic comparison solutions. A database approach is used by the COMDEF program to speed the calculation of comparison solutions, so that true dynamic backcalculation can be achieved quickly and accurately.

INTRODUCTION

Background

The use of nondestructive testing has been an increasingly cost effective tool in the evaluation of both airfield and highway pavements for the determination of structural condition. Results from nondestructive evaluations are used to estimate remaining life and allowable loads, as well as to provide data for design calculations.

A growing number of rigid pavements have been overlain with asphaltic concrete. These pavements have posed particular problems for evaluation and design. Many design procedures for overlays of rigid pavements, including the current Department of Defense [1] procedure for rigid airfield pavements, use an equivalent thickness method. The typical method is to compute a design thickness of Portland cement concrete pavement and estimate an overlay thickness of asphaltic concrete which will provide equivalent support. The total overlay thickness depends not only on the existing rigid pavement layer thickness, but also on condition factors which are determined from a visual inspection of the condition of the Portland cement concrete slabs. This method assumes that the condition of the rigid pavement layer can be determined. When a Portland cement concrete pavement has already had an asphaltic concrete overlay, so that the condition of the underlying rigid layer cannot be determined by visual inspection, improper determination of the rigid pavement properties can lead to premature failures (underdesign) or to increased costs (overdesign).

Recent developments in the nondestructive evaluation of pavements has led to the use of multilayer linear elastic modeling [2,3] to determine layer properties of composite pavements. However, the use of such a model requires an assumption of quasistatic behavior which ignores the dynamic nature of the nondestructive test. Dynamic behavior is of particular importance for vibratory devices such as the Road Rater or Dynaflect, because the use of steady-state vibration insures that significant dynamic effects will occur during the response measurements.

In the past, true dynamic evaluations of pavement systems have been limited in application due to the difficulty in obtaining true dynamic data and the large amount of computer time needed for theoretical dynamic comparison solutions. The methods presented herein provide a fast and accurate method for the data collection and reduction so that a true dynamic solution can be determined.

Backcalculation of Layer Moduli

The deformed shape of a pavement when subjected to a vertically oriented surface loading is commonly known as a deflection basin. In the case of a uniform circular load on a layered system, the theoretical deflection basin has the property of radial symmetry and is typically represented by a series of deflections along a radial axis. A nondestructive testing device, such as the Road Rater 2008, may apply a vibratory surface loading and measure the peak responses at radial distances from the center of the load. For an assumed ideal system with known properties, the responses at specific locations may be theoretically calculated for a given loading. The process of matching measured field data with the responses of an ideal system is known as backcalculation.

Idealized Modeling

All common methods of nondestructive evaluation of pavement systems utilize an idealized mathematical model for comparison with the real pavement system. Figure 1 illustrates some of the differences between a typical idealized model and a real pavement system. In general, the properties of the real and idealized systems are not the same. Use of nondestructive test data to backcalculate

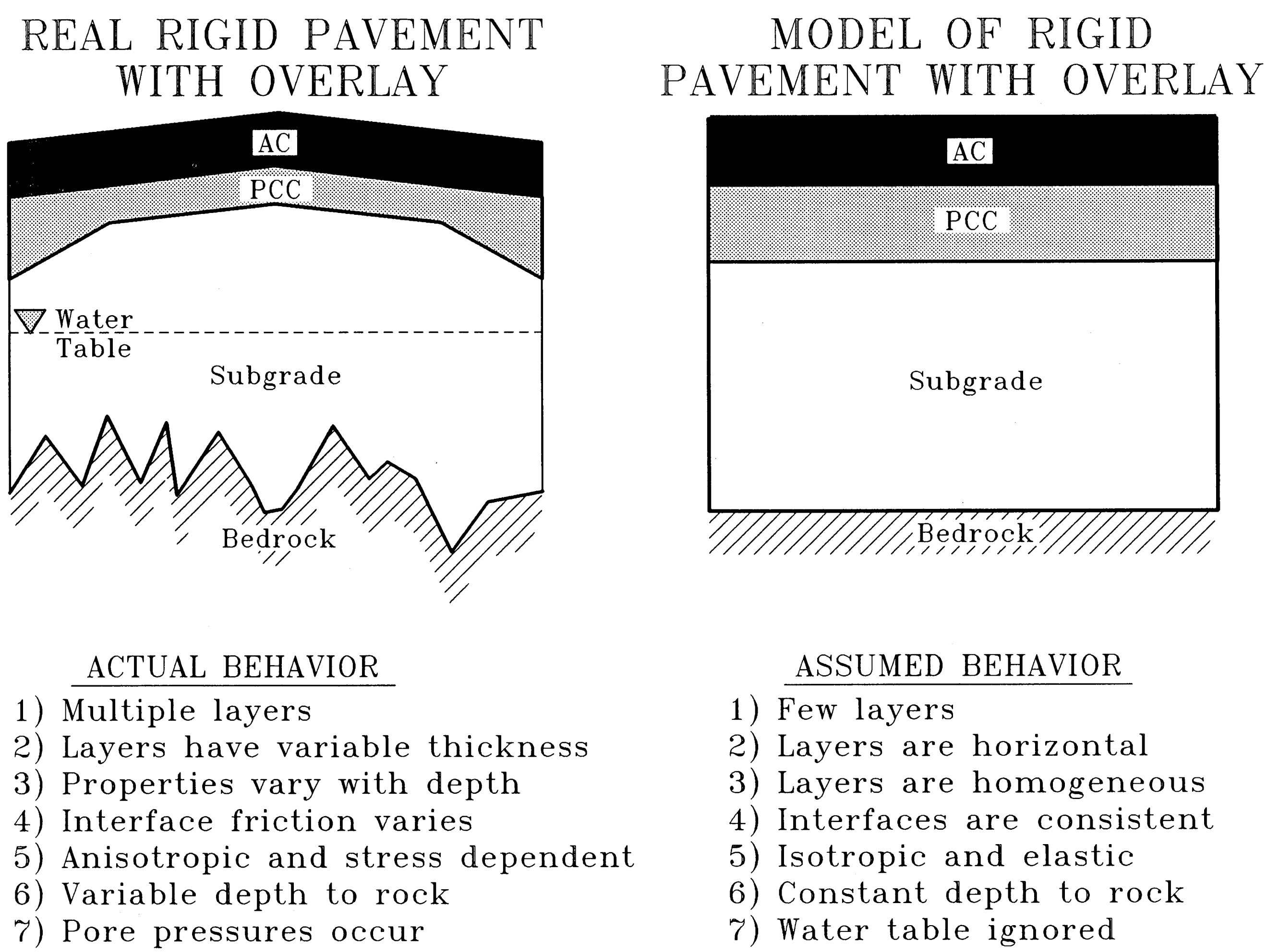

Figure 1. Idealized Modeling of Rigid Pavements with Overlays

layer properties implies that equivalent deflection responses are indicative of equivalent systems. In fact, this is not the case. Backcalculated moduli are effective moduli which apply *only* to the assumed idealized model. If layer properties from the idealized model are to be used in a subsequent design or remaining life prediction, it is essential that a consistent idealized modeling approach be used in the subsequent processes. As more sophisticated design procedures are developed and implemented, consistent evaluation models are required. One improvement to idealized modeling which has been considered is the use of a true dynamic model. Use of a true dynamic model can provide a more realistic model for comparison and subsequent design. However, the use of a dynamic model does not remove the need for consistency between evaluation and design procedures because the layer moduli backcalculated with a true dynamic model are still effective moduli which apply only to the assumed dynamic idealized model.

METHODOLOGY

Data Collection for Dynamic Analysis

The nondestructive testing device used to provide the dynamic loading for this study was a Road Rater 2008. An illustration of the trailer mounted vibratory device is shown in Figure 2. A schematic view of the Road Rater 2008 is included in Figure 3. The Road Rater 2008 is capable of generating vibratory loads of variable frequency and magnitude. For the purpose of this study, a frequency of 20 Hz and load magnitude of 7 kips (31 kN) were utilized. The test site for verification data was the Bomb Damage Repair Site (BDRS) at the US Army Engineer Waterways Experiment Station (WES) in Vicksburg, Mississippi. A site map and typical section are included in Figures 4 and 5, respectively.

Verification data were collected using the Road Rater 2008 in its standard configuration, except that the outputs from two of the geophones were run directly into a Hewlett-Packard 3562A Dynamic Signal Analyzer, which will be referred to herein as the Analyzer. Calibration factors for the Road Rater 2008 geophones at the frequency of interest were obtained. Time and frequency domain measurements were recorded by the Analyzer for subsequent analysis. Measurements were made at 1 foot (0.3 m) intervals from the center of the load plate using the second sensor location, 1 foot (0.3 m) from the plate center, as a reference location. Data of interest include the magnitudes of the sensor deflections at each location and the relative phase shift between each sensor location and the reference sensor.

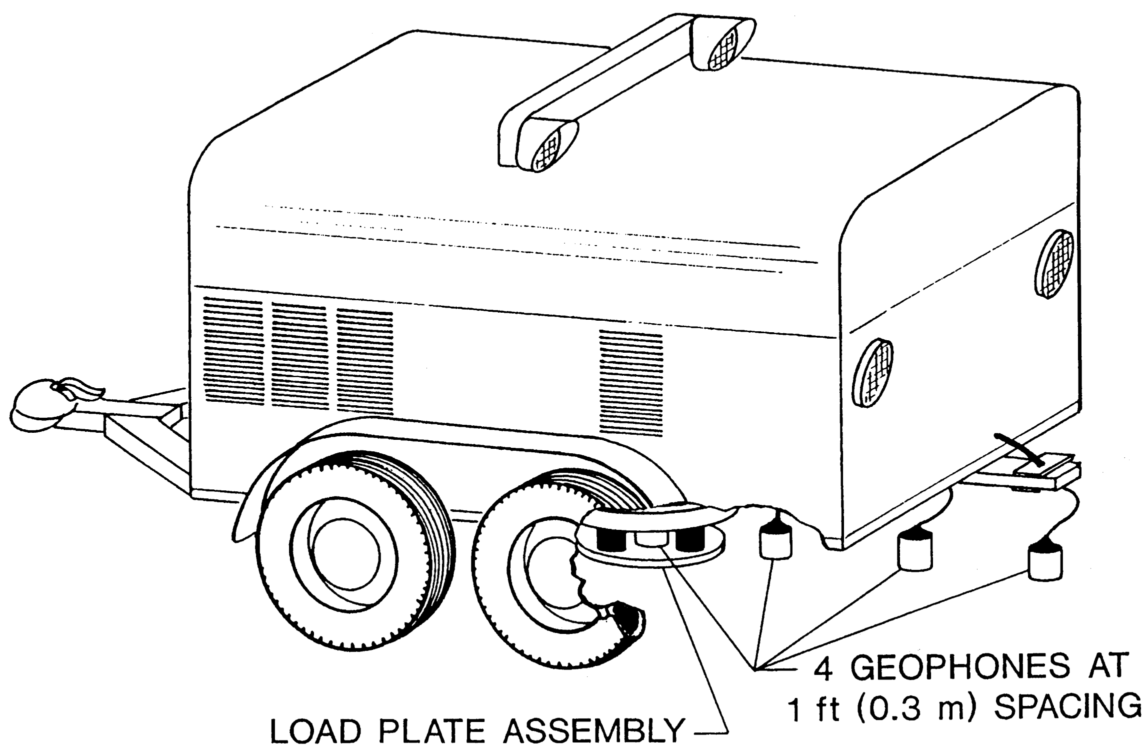

Figure 2. Road Rater 2008

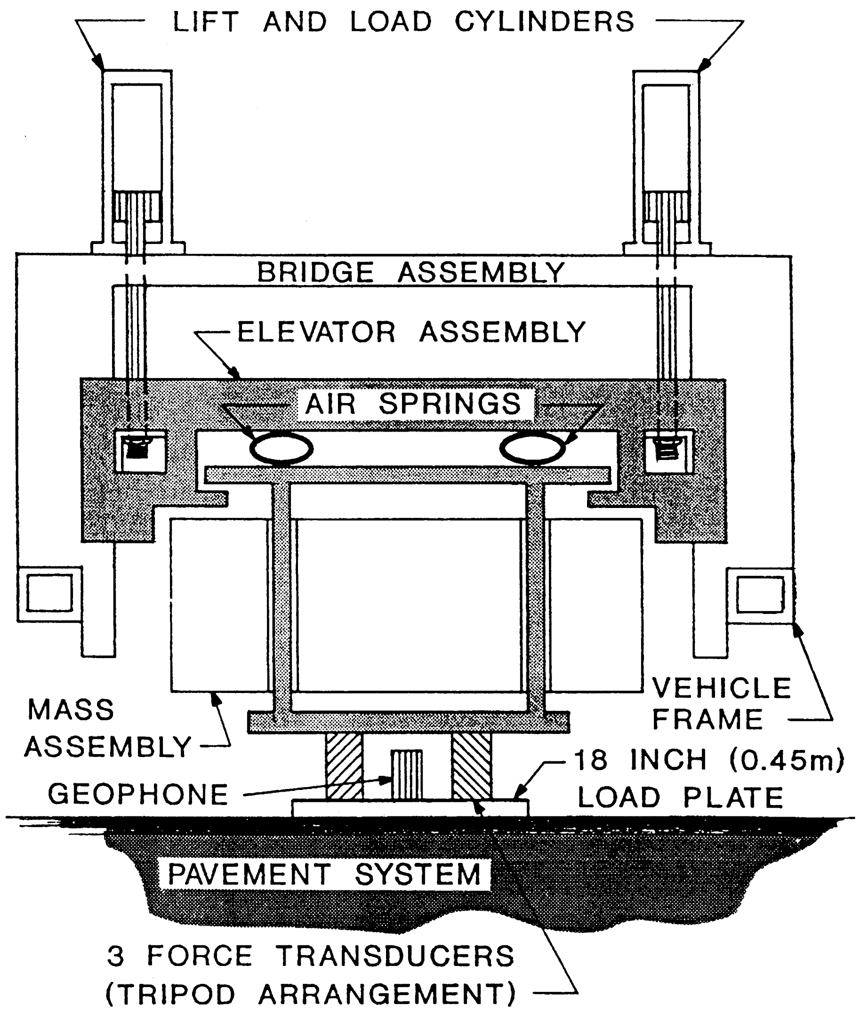

Figure 3. Schematic of Road Rater 2008 System

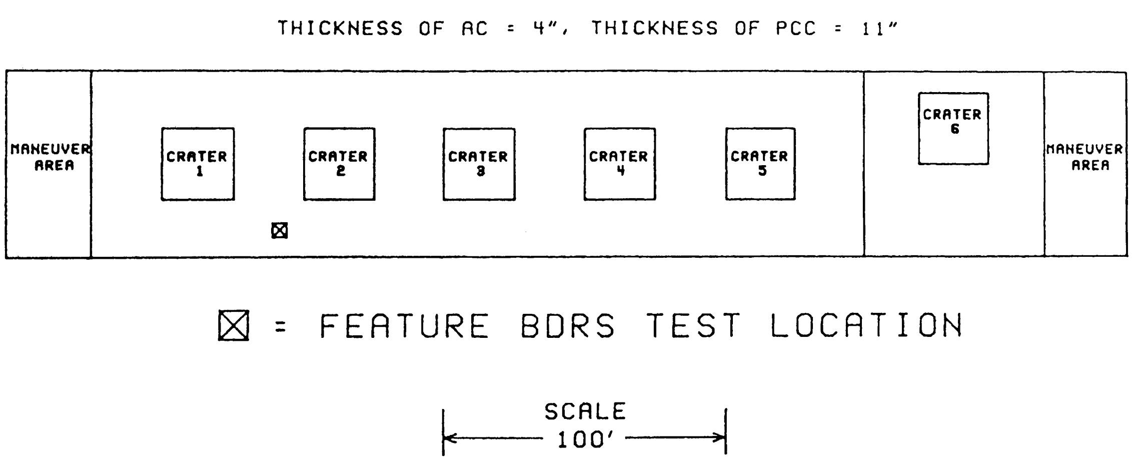

Figure 4. Site Map of WES Bomb Damage Repair Site (BDRS)

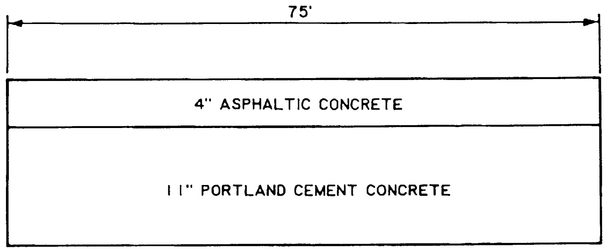

Figure 5. Typical Section for WES BDRS

The cross spectrum function of the Analyzer was used to record the phase shift between the two sensors. Figure 6 illustrates the measurement of relative phase shift as measured by the Analyzer.

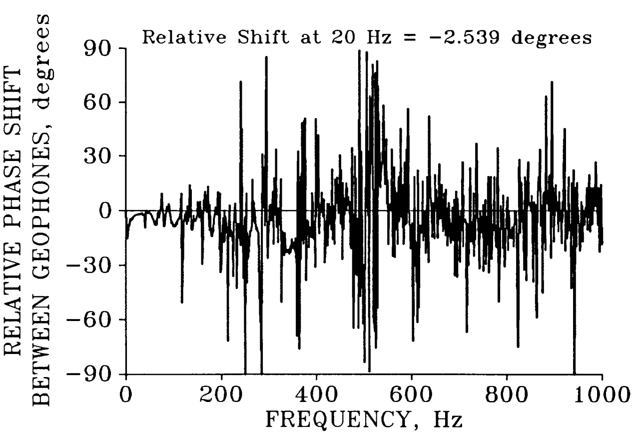

Figure 6. Relative Phase Shift from Cross Spectrum (Typical)

Fast Fourier Transform (FFT) analysis was used to determine the magnitude of the deflection at each sensor location from time domain recordings. The Road Rater 2008 recording system was used to monitor the load. Although the load measurement was not precisely synchronized with the response measurements, at each test location the ratio of the sensor response to the corresponding reference sensor response is relatively unaffected by load fluctuations. To minimize any errors in deflection magnitude due to load fluctuations for the six response measurements, the following procedure was used: (1) time domain waveforms of voltage versus time were recorded, as illustrated in Figure 7, (2) calibration factors were applied to obtain velocity versus time waveforms, as illustrated in Figure 8, (3) the FFT of each waveform was performed to obtain the frequency domain spectrum of velocity magnitude, as illustrated in Figure 9, (4) the velocity was integrated to obtain the deflection using division by -$j\omega$ (assumes a single

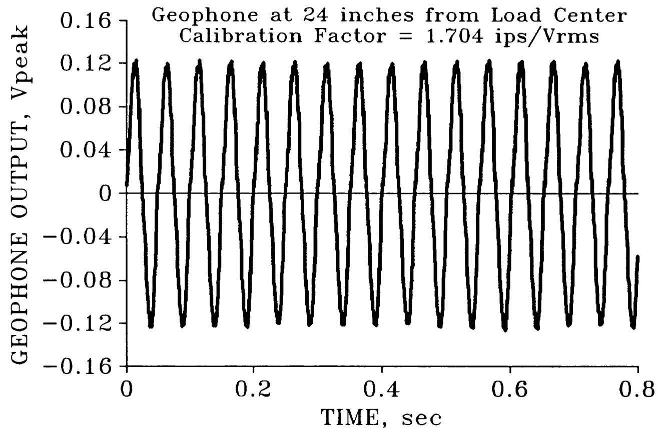

Figure 7. Geophone Output in Time Domain (Typical)

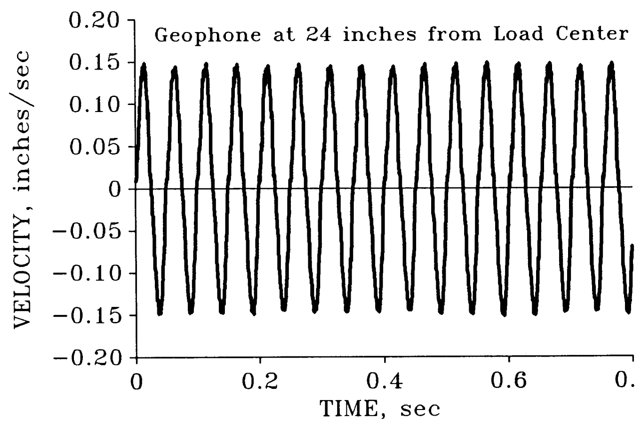

Figure 8. Geophone Velocity in Time Domain (Typical)

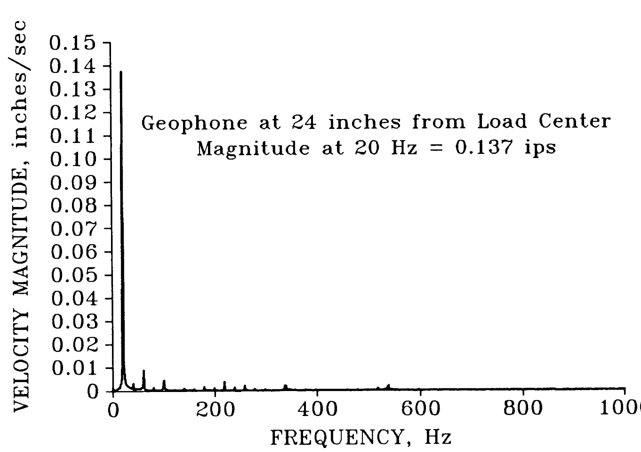

Figure 9. Geophone Velocity Magnitude in Frequency Domain (Typical)

degree of freedom system under sinusoidal loading), as illustrated in Figure 10, (5) the deflection magnitude spectrum at each location was divided by the corresponding deflection magnitude spectrum of the reference sensor, obtaining a ratio analogous to a transfer function value, as illustrated in Figure 11, (6) the average magnitude of the reference deflection for the six tests was assumed to be the value corresponding to a 7 kip (31 kN) loading, and (7) the magnitudes of deflection at each sensor location were obtained by multiplying each of the "transfer function" ratios by the average value of reference deflection. This procedure minimized the deleterious effect of load level fluctuations, resulting in measurements similar to that which would be measured if the data for seven channels were taken simultaneously. Figures 7-11 were provided for illustration and represent waveform analyses which were performed directly using Analyzer functions.

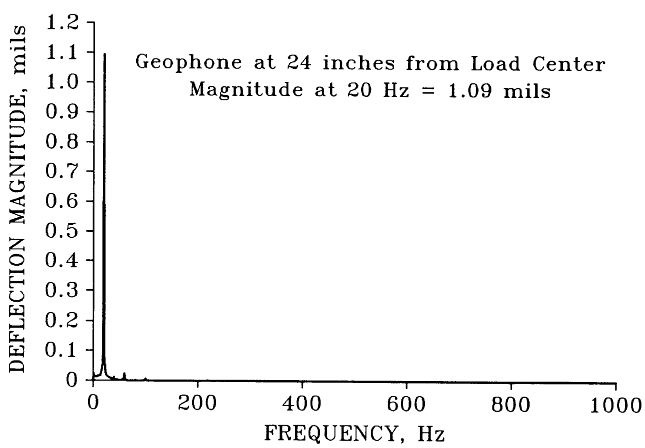

Figure 10. Geophone Deflection Magnitude in Frequency Domain (Typical)

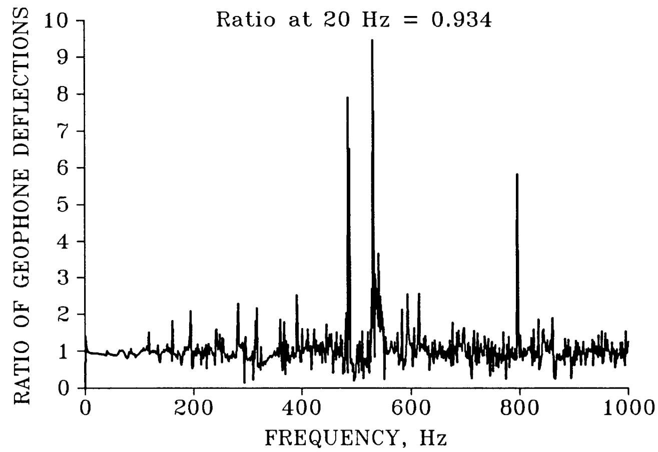

Figure 11. "Transfer Function" Ratio of Deflections (Typical)

Table 1 summarizes the raw data described above, including the peak deflection magnitudes from the FFT analyses and the phase shift data from the cross spectrum analyses. Table 2 summarizes data calculated from the raw data presented in Table 1. "Real time" deflection basins may be calculated using the magnitude and phase information. A real time basin is the equivalent of a "photograph" of the dynamic deflection basin at a given instant of time. The data presented in Table 2 are for a real time basin at the instant that the first sensor has reached its peak. Figure 12 illustrates the concept of the real time deflection basin. Each of the basins shown is a "photographic" image of the deflection basin at a given instant. The deflection basin typically reported is actually the envelope of peak values, as shown in Figure 12. As can be seen by inspection, the peak values of deflection do not occur at the same time. Both peak deflections and real time deflections will be used in backcalculation schema in following sections.

TABLE 1. Road Rater 2008 Raw Data

Sensor Location	Peak Reference Deflection from FFT	Peak Sensor Deflection from FFT	Ratio of Deflection to Reference	Phase Shift Relative to Reference	Phase Shift Relative to First Sensor
0 inches	1.197152	1.539015	1.285564	-7.7578	0.0000
12 inches	----	---	1.000000	0.0000	7.7578
24 inches	1.170470	1.093391	0.934147	2.5390	10.2968
36 inches	1.186607	0.990091	0.834388	8.2780	16.0358
48 inches	1.195363	0.924818	0.773671	14.2900	22.0478
60 inches	1.141039	0.786801	0.689548	21.5100	29.2678
72 inches	1.179272	0.772117	0.654740	30.4130	38.1708

Table 2. Dynamic Deflection Basins for the Road Rater 2008

Sensor Location	Average 7-kip Deflection of Reference	Ratio of Deflection to Reference	Peak Sensor Deflection Magnitude	Phase Shift Relative to First Sensor	Real Time Deflection Basin
0 inches	1.178317	1.285564	1.514802	0.0000	1.514802
12 inches	1.178317	1.000000	1.178317	7.7578	1.167533
24 inches	1.178317	0.934147	1.100721	10.2968	1.082994
36 inches	1.178317	0.834388	0.983174	16.0358	0.944918
48 inches	1.178317	0.773671	0.911630	22.0478	0.844963
60 inches	1.178317	0.689548	0.812506	29.2678	0.708785
72 inches	1.178317	0.654740	0.771491	38.1708	0.606525

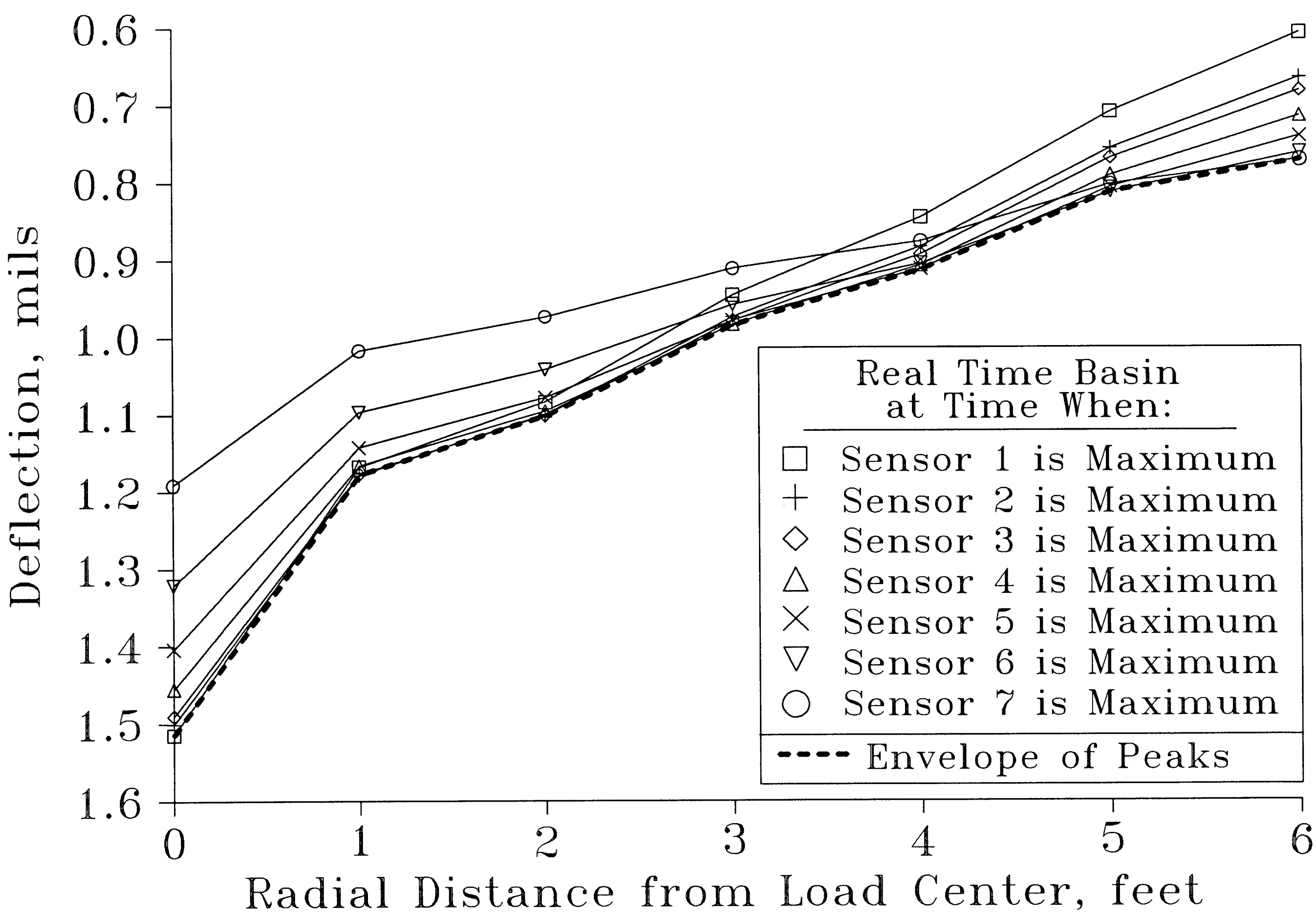

Figure 12. Real Time Deflection Basins

Dynamic Response Predictions by Green's Functions

Dynamic structural response predictions for layered media were pioneered by Haskell [4] and Thomson [5]. The Haskell-Thomson method uses transfer matrices to solve the problem in the frequency domain. For arbitrary loadings, the time domain loading function is resolved into a series of harmonic vibrations, typically by the Fast Fourier Transform. This is equivalent to the use of the method of separation of variables to solve the wave equation. Closed form solutions are possible for simple cases, but numeric techniques are required for arbitrarily layered systems. Kausel and Roesset [6] developed an approach which used a stiffness matrix to relate external loads to displacements, where the stiffness matrix contains functions of frequency and wave number. This formulation was inefficient, because the arguments become transcendental functions which can be solved in closed form only for very simple geometries. If the layer thicknesses are small compared to the wavelengths of interest, it is possible to reduce the transcendental equations to a series of algebraic expressions. This procedure was first proposed by Lysmer and Wass [7] and later generalized by Wass [8] and Kausel [9]. An improved method for the use of Green's functions in dynamic structural response predictions, developed by Kausel [10], allowed an explicit closed form solution for the response of a layered medium to dynamic loads. A cursory description of Kausel's method is presented herein.

For the nondestructive testing of pavements, the problem of interest is the case of a vertical disc load on a layered medium, and discussion of the method will be restricted to this special case. If a specific layer, m, is isolated and equilibrium preserved, the relationship between external loads and displacements may be written as:

$$P = K_m U \qquad (1)$$

where

P = external load vector
K_m = stiffness matrix
U = displacement vector

For a multilayered system, the global stiffness matrix, $K = \{K_m\}$, is constructed by superimposing the contribution of the layer matrices at each interface. For the case of thin layers [6], the layer stiffness matrix may be written as:

$$K_m = A_m k^2 + B_m k + G_m - \omega^2 M_m \qquad (2)$$

where

k = wave number
ω = frequency of excitation
A_m, B_m, G_m, M_m = matrices depending only on layer properties

For this formulation, the stiffness matrices are functions only of the layer material properties, and the relationships are algebraic. The global stiffness matrix is formed by superimposing the layer stiffness matrices, and the displacements could be obtained by the formal inversion of the stiffness matrix, that is:

$$U = K^{-1} P \qquad (3)$$

Formal inversion is not necessary for the problem of interest. The natural modes of wave propagation are obtained from the eigenvalue problem produced by setting the load vector equal to zero:

$$(Ak_j^2 + Bk_j + G - \omega^2 M)\ \phi_j = 0 \qquad (4)$$

The notation for the displacement vector has been changed from U to ϕ_j to identify it as an eigenvector. The problem yields $6N$ eigenvalues, k_j, and eigenvectors, ϕ_j, where N is the total number of layers. Half of the solution set corresponds to k_j, ϕ_j. The other half of the solution set corresponds to $-k_j$, ϕ_j^*, with ϕ_j^* being obtained trivially from ϕ_j by reversing the sign of the vertical components. Following Wass [8], the $3N$ modes that decay with distance from the source, or propagate away from the source, are considered. These correspond to eigenvalues k_j, whose imaginary part is negative if k_j is complex, or whose real part is positive if k_j is real. Solution of the quadratic eigenvalue problem as a linear double dimension problem is not necessary because the special structure of the matrices involved allows the rows and columns to be rearranged by degrees of freedom rather than by interface. For the special case of the uniform vertical disk load, the $2N$ modes which represent Rayleigh wave propagation are included and the N modes which represent Love wave propagation may be ignored. By this method, the solution can be expressed as:

$$u_z = qR \sum_{\ell=1}^{2N} \phi_z^{m\ell} \phi_z^{n\ell} I_{1\ell}^R \qquad (5)$$

where

q = load intensity
R = vertical disk radius
m = interface for displacements
  = 1 at the surface
n = interface for disk load
  = 1 at the surface
ℓ = Rayleigh modes = 1, 2, ... , 2N
$I_{1\ell}^R = (\pi/2ik_\ell)J_0(k_\ell\rho)H_1^2(k_\ell R) - (1/Rk_\ell^2)$ for $0 \le \rho \le R$
$\quad = (\pi/2ik_\ell)J_1(k_\ell R)H_0^2(k_\ell\rho)$ for $R \le \rho$
$i = (-1)^{1/2}$
J_ς = Bessel function of the first kind and order ς
H_ς^η = Hankel function of the η^{th} kind and order ς

A Dynamic Backcalculation Algorithm

A backcalculation algorithm called COMDEF [3] was utilized in this study for dynamic backcalculations. The original version of COMDEF used multilayer linear elastic (quasistatic) theory as its structural response model and utilized a database method to improve the speed and accuracy of the backcalculations. The COMDEF program provides a good framework for backcalculation and was designed so that a more complicated structural response model could be substituted simply by substituting new database files. A theoretical method for making dynamic structural response predictions of the deflections under nondestructive loading was utilized to prepare a demonstration database for dynamic backcalculation. The dynamic method uses the Green's function method previously described to predict dynamic structural responses [10] and was implemented in the microcomputer program GREEN-MA [11]. A major disadvantage of the Green's function method developed by Kausel and implemented in GREEN-MA is the computer time required to make structural response predictions. Use of the COMDEF program provided the speed of calculation needed for true dynamic backcalculations. A flow chart of COMDEF is shown in Figure 13. Typical COMDEF data and output files are shown in Figure 14.

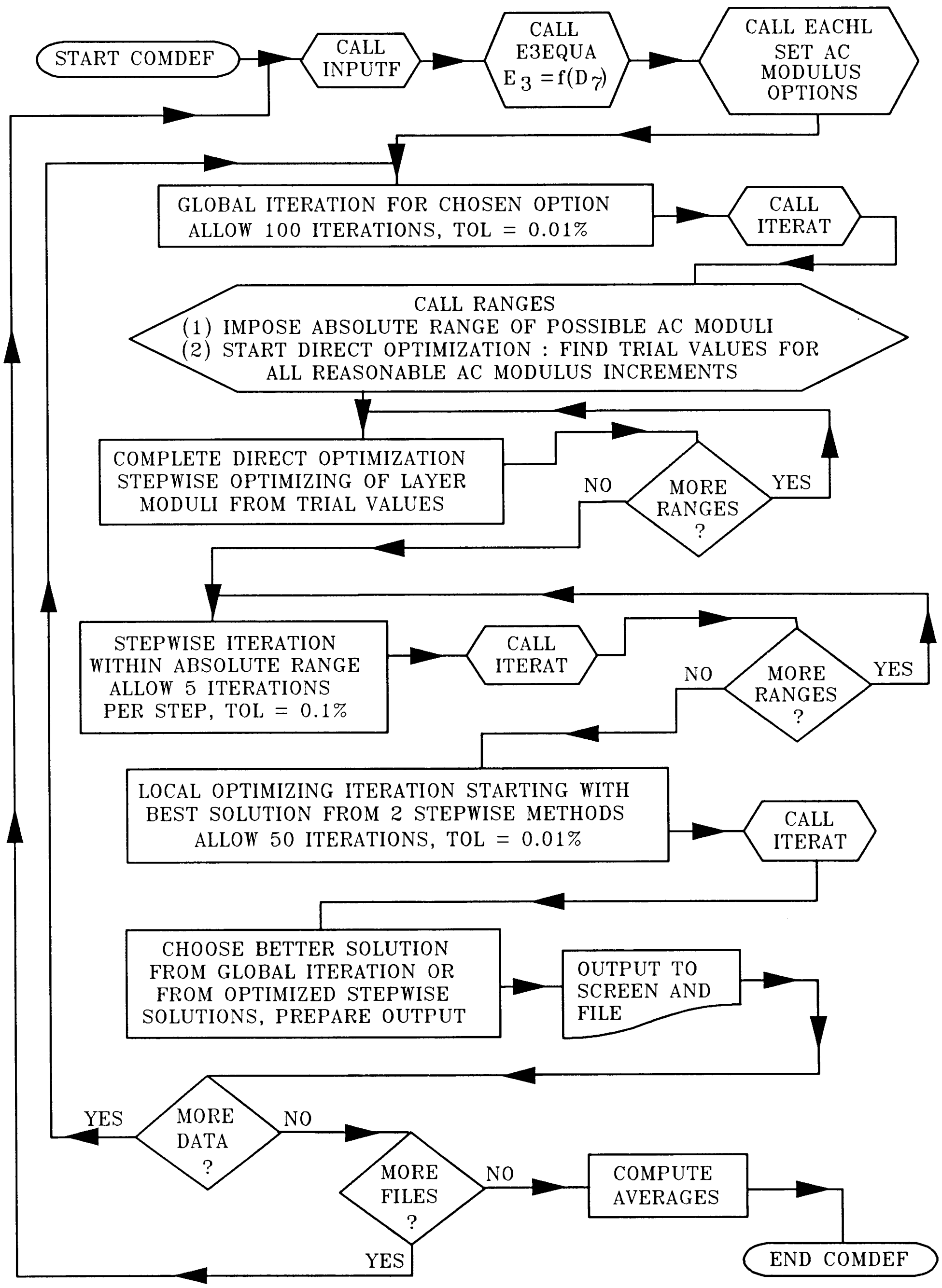

Figure 13. Flow Chart of COMDEF Program

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PAVEMENT FACILITY OR FEATURE ID: EXAMPLE DATA
 1 NDT LOADINGS PER TEST LOCATION
STATION TRACK  DATE  TEMP LOAD   D1   D2   D3   D4   D5   D6   D7
******* ***** ****** **** ***** **** **** **** **** **** **** ****
    1.0    1  870421 70.0 25025 20.9 18.1 16.2 14.1 12.0 10.0  8.2
```

```
PAVEMENT FACILITY OR FEATURE ID: EXAMPLE DATA


SOLUTION FOR PROBLEM   1 OF   1 FOR FILE example.out    :


STATION NUMBER =      1.00
TRACK NUMBER =           1
DATE OF TEST =      870421
SURFACE TEMPERATURE =  70.0 DEGREES F
THICKNESS OF AC =  6.00 INCHES
THICKNESS OF PCC =  7.00 INCHES
DYNAMIC LOAD = 25025. POUNDS

MODULUS OF AC        =     556208. PSI
MODULUS OF PCC       =    5090788. PSI
MODULUS OF SUBGRADE =        7975. PSI

            DISTANCE     ACTUAL     PREDICTED
 SENSOR    FROM LOAD  DEFLECTION  DEFLECTION
 NUMBER    (INCHES)     (MILS)      (MILS)
   1          0.        20.90       20.90
   2         12.        18.10       18.10
   3         24.        16.20       16.23
   4         36.        14.10       14.09
   5         48.        12.00       11.97
   6         60.        10.00        9.99
   7         72.         8.20        8.22


SUM OF ABSOLUTE VALUE OF ERRORS IN DEFLECTION =    0.11 MILS
TOTAL PERCENTAGE ERROR IN DEFLECTION BASIN =    0.88 %
```

FIGURE 14. Example Data File and Output File for COMDEF Program

Backcalculation Schema

The main purpose of the dynamic backcalculation analyses presented herein is to demonstrate the importance of a true dynamic method. However, during the analysis, an attempt was made to demonstrate various levels of implementation of the dynamic methodology. To achieve this additional goal, several backcalculation schema were devised. These schema are summarized in Table 3, and generally represent increased cost of implementation to the end user. The five backcalculation schema include:

(1) <u>STD</u> A "standard" method which represents the typical backcalculation method currently used in practice for nondestructive evaluations with the Road Rater 2008. This method measures peak deflection measurements from 4 sensors at 1 foot (0.3 m)

TABLE 3. Backcalculation Schema for True Dynamic Method

Schema Identification	Sensor Locations Used	Phase Data Used	Forward Model
STD	0, 12, 24, 36 inches	None	Linear Elastic
4D-OP	0, 24, 48, 72 inches	None	Dynamic
4D-3P	0, 24, 48, 72 inches	Relative Phase Shift	Dynamic
7D-PK	0, 12, 24, 36, 48, 60, 72 in.	None	Dynamic
7D-RT	0, 12, 24, 36, 48, 60, 72 in.	Real Time Basin	Dynamic

intervals. These deflection measurements are analyzed by a backcalculation algorithm which utilizes multilayer linear elastic (quasistatic) theory as its structural response model.

(2) 4D-OP A dynamic method which represents the minimum investment by the user. In this method, the standard Road Rater 2008 sensor arrangement is utilized and the standard 4 peak deflections are measured. However, for this case a dynamic structural response model is substituted for the quasistatic model typically used in practice. Therefore the dynamic peak values measured by the Road Rater are compared to true dynamic peak values.

(3) 4D-3P A dynamic method representing relatively minor modifications of the Road Rater 2008 system. For this schema, the spacing of the 4 standard Road Rater 2008 sensors is increased to 2 foot (0.6 m) intervals. In addition, a spectrum analyzer is incorporated to measure phase shifts. In this schema, the 4 peak deflections and the 3 relative phase shifts between sensors are used as 7 inputs to a true dynamic backcalculation algorithm.

(4) 7D-OP A dynamic method requiring significant modifications to the Road Rater 2008 but not requiring a spectrum analyzer. In this schema, the number of sensors is increased to 7 and the peak deflections are used as 7 inputs to a true dynamic backcalculation algorithm.

(5) 7D-RT A dynamic method which represents the complete implementation by the end user. In this schema, a real time basin is measured and used as input to a true dynamic backcalculation algorithm.

Formulation of Database Files

A matrix of 210 precalculated solutions is required to form a constants file for use in the COMDEF program, as indicated in Table 4. Structural response predictions were made with GREEN-MA for the 210 cases of interest. A single set of 210 solutions provided the data to create the databases for all 4 of the dynamic backcalculation schema, since each of the 210 GREEN-MA runs provided the complex Green's functions for each of the 7 sensor locations. In addition to the dynamic structural response solutions, a set of 210 quasistatic solutions were used in the "standard" schema (STD).

All backcalculations performed in this study used the standard COMDEF program without modification. That is, the different backcalculation schema used the same executable file (COMDEF.EXE) and differed only in the database files utilized. The COMDEF program expects 7 deflections and corrects the deflections to a standard load. Since some of the schema utilized data which did not meet this criteria, some special manipulations were used in the creation of the database files and in the creation of the data files to insure that the backcalculation was completed properly without requiring changes to the COMDEF program.

TABLE 4. Variable Matrix[a] for Database Files[b]

Variable Modulus	Matrix Values (ksi)
AC	33, 82, 205, 512, 1280, 3200
PCC	82, 205, 512, 1280, 3200, 8000, 20000
Subgrade	2, 6, 18, 54, 162

[a] Deflections are calculated for each combination of the variable matrix at seven sensor locations located on 1 foot centers along a line measured from the center of the load plate (centers at 0, 12, 24, 36, 48, 60, and 72 inches).

[b] Total number of deflections per database file is equal to the product of seven sensor locations times six moduli of AC times seven moduli of PCC times five moduli of subgrade for a total of 1470 deflections per file.

Depth to Bedrock as a Controlling Factor

Assumption of a rigid layer at a depth of 20 feet (6 m) is common for backcalculation using multilayer linear elastic theory [12]. This assumption has been used in the past for dynamic structural response predictions [13]. However, this assumption should not be used in a dynamic backcalculation method unless the depth to rigid material is actually 20 feet (6 m). Just as system resonance is a function of frequency for a fixed pavement modulus profile, system resonance becomes a function of the pavement modulus profile for a fixed frequency of test. For the case of vibratory loading at 20 Hz, and for the assumption of a rigid layer at 20 feet (6 m), the predicted system resonance occurs when the modulus of subgrade is about 50 ksi (0.3 GPa). This predicted resonance will bias the backcalculation so that reasonable moduli values, even those far from the predicted resonant condition, cannot be found routinely. Based on a cursory review of the resonance condition described above, it is concluded that an assumed depth to rigid boundary must be at least 60 feet (18 m) to avoid the predicted resonant condition for a reasonable range of subgrade moduli. For the dynamic analyses included in this study the approximate actual depth to rock of 133 feet (41 m) was assumed. This depth is based on a boring completed by WES researchers at a nearby site on March 21, 1985, as summarized in Figure 15.

Role of Material Damping in Dynamic Calculations

Geometric, or radiation, damping due to the vibratory loading of a massless disc is taken into account automatically in the method developed by Kausel, as implemented in the program GREEN-MA. Additional damping due to the Road Rater mass assembly was neglected. Material damping must be assumed for structural response calculations by this method. Numerous authors have shown that the upper limit for material damping in soils is about 10% [14]. It is common to assume a value of 5% for the material damping of all pavement layers for dynamic calculations [13]. It has been reported that the choice of assumed material damping ratio has a minor effect in comparison with the geometric damping associated with wave propagation in a continuum for nondestructive testing of pavements [13,15,16]. A set of structural response predictions was used to evaluate this effect, and the calculations supported this conclusion. Therefore, all dynamic calculations reported herein use the assumption of material damping equal to 5%.

RESULTS OF DYNAMIC BACKCALCULATIONS

The results of the dynamic backcalculations are summarized in Figures 16 and 17. The asphaltic concrete modulus was assigned based on results of

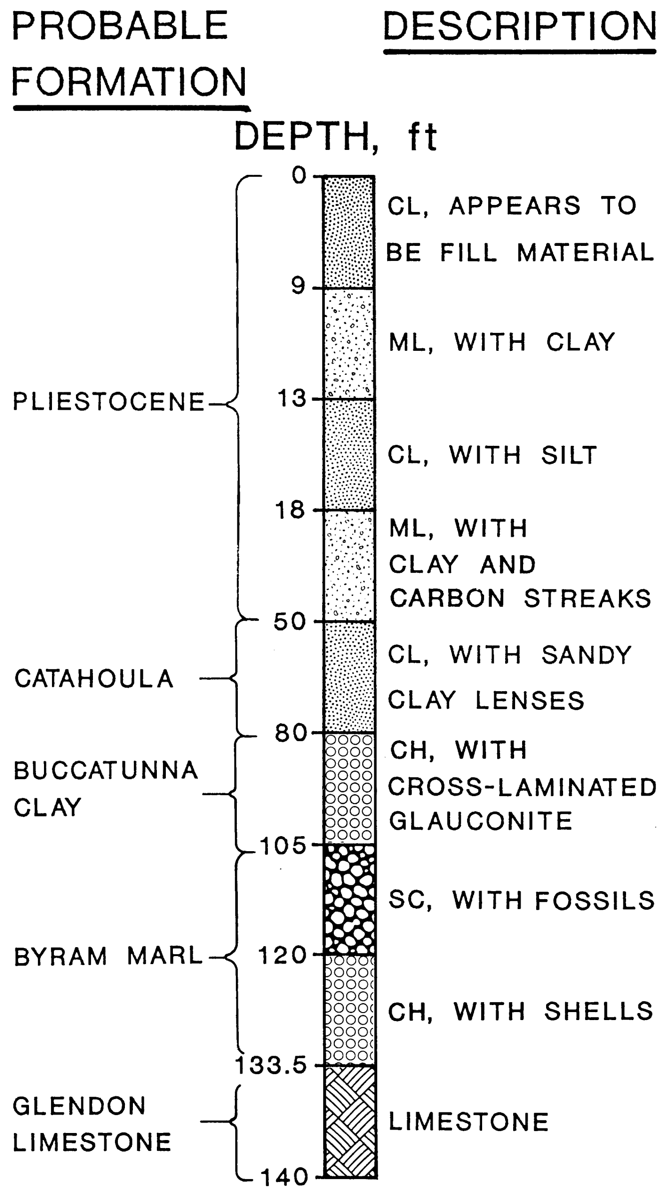

Figure 15. Soil Exploration Near the BDRS Site

laboratory resilient modulus tests on asphaltic concrete cores from the BDRS site, using the Kentucky method [17] to estimate mean asphalt temperature at the time of the testing.

Figure 16 compares the values of backcalculated Portland cement concrete moduli for the various schema with the average laboratory modulus value based on fundamental frequency and wave propagation tests. The quasistatic calculation used in schema STD proved inadequate in this case, backcalculating a very low modulus value. All of the dynamic backcalculation schema provided reasonable solutions, with the best solutions for schema 4D-3P and 7D-RT, which utilized phase data.

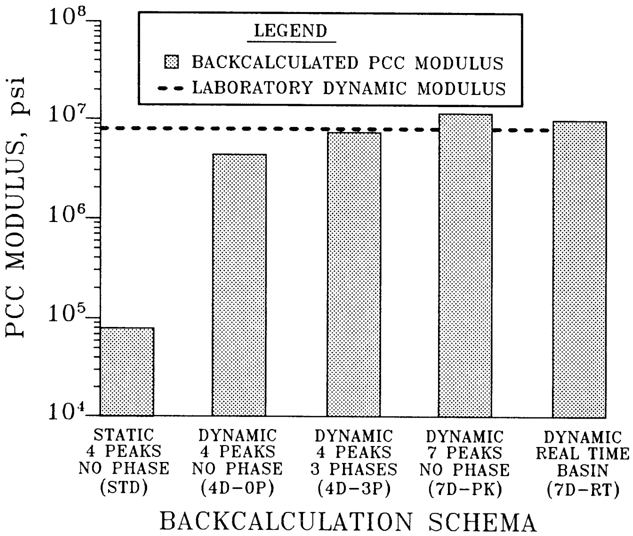

Figure 16. Backcalculated PCC Modulus

Figure 17 compares the values of backcalculated subgrade moduli for the various schema with an expected range of subgrade moduli. The expected range was determined on a subjective basis from personal communication with Dr Albert J. Bush of the Waterways Experiment Station. Dr Bush has, over a period of years, performed extensive testing at (or near) the BDRS site.

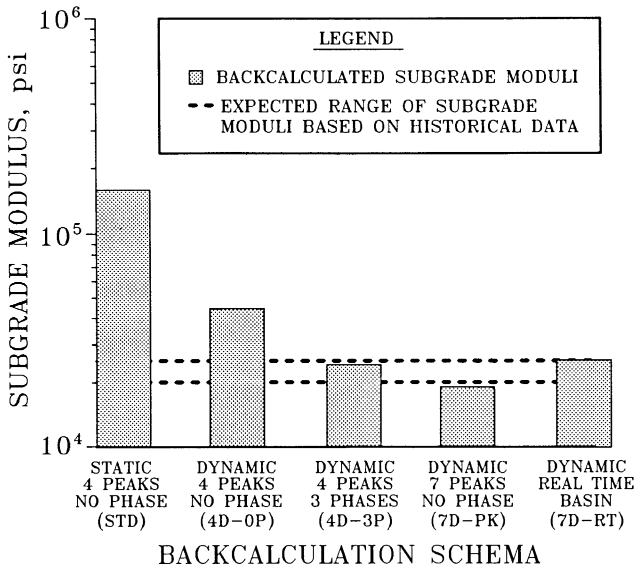

Figure 17. Backcalculated Subgrade Modulus

The quasistatic calculation used in schema STD was inadequate for the subgrade modulus, predicting an unrealistically high value. All of the dynamic solutions except schema 4D-0P gave reasonable values for subgrade moduli. This points out that 4 peak deflections, even with a dynamic solution, may not give good results for backcalculated subgrade moduli unless more data are included either in the form of phase data or more deflection data. As with the Portland cement concrete modulus, the results were reasonable when phase data was included. These results help explain the difficulties some researchers [*e.g.*, 13] have encountered when attempting dynamic backcalculation without phase data.

CONCLUSIONS AND RECOMMENDATIONS

Based on the data presented herein, it is concluded that the use of a true dynamic model can improve the accuracy and applicability of backcalculated pavement parameters. Use of phase data is strongly recommended in the future for data collection and analysis for nondestructive testing, particularly with steady-state vibratory devices. Also, based on the results from the different backcalculation schema, it appears that a significantly improved solution can be achieved with a relatively small investment in hardware and software. The signal analyzer used in this study was relatively expensive, but was available for this project and had the advantage of automating much of the data analysis. However, recent advances in data acquisition technology have led to reasonably priced state-of-the-art signal acquisition and analysis hardware and software for microcomputers (*i.e.*, MS-DOS compatible), which are recommended.

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REFERENCES

[1] Department of Defense, *Rigid Pavements for Airfields*, Technical Manual TM-5-825-3/AFM-88-6, Ch. 3, Washington, DC, 1986.

[2] Bush, A.J., III and Alexander, D.R., "Pavement Evaluation Using Deflection Basin Measurements and Layered Theory," *Transportation Research Record 1022*, Transportation Research Board, National Research Council, Washington, DC, 1985.

[3] Anderson, M., "A Database Method for Backcalculation of Composite Pavement Layer Moduli," *ASTM STP-1026*, The First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, American Society for Testing and Materials, Baltimore, Maryland, 1988.

[4] Haskell, N.A., "The Dispersion of Surface Waves in Multilayered Media," *Bulletin of the Seismological Society of America*, Vol. 43, No. 1, 1953.

[5] Thomson, W.T., "Transmission of Elastic Waves Through a Stratified Solid Medium," *Journal of Applied Physics*, Vol. 21, February 1950.

[6] Kausel, E. and Roesset, J.M., "Stiffness Matrices for Layered Soils," Vol 71, No 6, *Bulletin of the Seismological Society of America*, December 1981.

[7] Lysmer, J. and Waas, G., "Shear Waves in Plane Infinite Structures," Vol 98, No EM1, *Journal of the Engineering Mechanics Division*, ASCE, February 1972.

[8] Waas, G., *Linear Two-Dimensional Analysis of Soil Dynamics Problems in Semi-infinite Layer Media*, Ph.D. Dissertation, University of California, Berkeley, California, 1972.

[9] Kausel, E., *Forced Vibrations of Circular Foundations on Layered Media*, Ph.D. Dissertation, Department of Civil Engineering, M.I.T., Cambridge, Massachusetts, 1974.

[10] Kausel, E., "An Explicit Solution for the Green Functions for Dynamic Loads in Layered Media," Publication No. R81-13, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1981.

[11] Anderson, M., *Backcalculation of Composite Pavement Layer Moduli*, Ph.D. Dissertation, University of Kentucky, Lexington, Kentucky, 1988.

[12] Bush, A.J., III, "Nondestructive Testing for Light Aircraft Pavements, Phase II, Development of the Nondestructive Evaluation Methodology," Report No. FAA-RD-80-9-II, Geotechnical Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1980.

[13] Mamlouk, M.S., "Use of Dynamic Analysis in Predicting Field Multilayer Pavement Moduli," *Transportation Research Record 1043*, Transportation Research Board, National Research Council, Washington, DC, 1985.

[14] Richart, F.E., Jr., Hall, J.R., Jr., and Woods, R.D., *Vibrations of Soils and Foundations*, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1970.

[15] Roesset, J.M. and Shao, K.-Y., "Dynamic Interpretation of Dynaflect and Falling Weight Deflectometer Tests," Transportation Research Record 1022, Transportation Research Board, National Research Council, Washington, DC, 1985.

[16] Davies, T.G. and Mamlouk, M.S., "Theoretical Response of Multilayer Pavement Systems to Dynamic Nondestructive Testing," *Transportation Research Record 1022*, Transportation Research Board, National Research Council, Washington, DC, 1985.

[17] Southgate, H.F. and Deen, R.C., "Temperature Distribution within Asphalt Pavements and Its Relationship to Pavement Deflection," *Highway Research Record 291*, Highway Research Board, Washington, DC, 1969.