sidered obsolete. The vibrational levels observed in taxiing aircraft are well below the comfort levels shown in the AMRL criteria, and conservative results are indicated. However, there is no other information available about human response to shock or psychologically alarming effects at such low levels of vibration.

The effects of runway roughness on aircraft structural fatigue and avionics failure were investigated briefly. However, criteria based on these subjects, in addition to human response, rely on aircraft vibration response. The use of aircraft response as a primary tool for establishing roughness criteria is subject to question, since it is an indirect approach to the problem. Thus, the more direct approach of analyzing the profiles rather than the responses was pursued.

RESULTS

The overall displacement roughnesses of 21 available profiles are given below (1 mm = 0.039 in).

Airport	Runway	σ(mm)
Clovis, New Mexico	03	2.899 4
Palmdale, California	07	3.131 6
Dallas-Ft. Worth	17R	3.694 7
Dulles International, Virginia	01	3.821 9
Edwards Air Force Base, California	04	4.050 5
Grand Forks, North Dakota	35	4.277 4
Chicago-O'Hare	32L	4.429 3
Chicago-O'Hare	27L	5.430 0
Oklahoma City	TE	5.645 9
Charleston, South Carolina	15	6.630 4
Oklahoma City	12	6.753 6
Albuquerque	17	6.951 5
Baltimore-Washington International	28	7.067 0
Newark, New Jersey	22L	7.232 7
Offutt Air Force Base, Omaha, Nebraska	12	7.410 2
Chicago-O'Hare	22L	7.414 8
John F. Kennedy, New York	13R	7.674 1
Buffalo	05	9.218 7
Washington National	18	9.460 2
Albany	19	11.272 5
Thule	16	13.400 0
Avg.		7.082 0

The following general conclusions and observations have been made.

1. Runway 03 at Clovis is the smoothest runway.
2. Runway 16 at Thule is the roughest runway.
3. Runway 28 at Baltimore-Washington represents the average runway.
4. Although runway TE at Oklahoma City has an above average overall index, there is one severe bump that is probably in need of local repair, but only a statistical analysis can determine this quantitatively.

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Runway-roughness-rating methods based on aircraft-response properties are not recommended. These methods consider structural, component, or human-failure mechanisms and cannot be correlated precisely with local bumps that need repair. Knowledge of aircraft responses can be useful for evaluating the effectiveness of a repair plan, but cannot be used in constructing the repair plan. The response data can also be analyzed statistically to obtain information about amplitude and frequency content for studying aircraft vibration. The collection of such data is recommended, but not as the primary information on which to base a repair plan.

The statistical analysis of filtered profile data is recommended as a direct method of assessing runway profile roughness. The approach described here is only a beginning, as only the displacement standard deviation has been extracted as a primary index of roughness. The overall properties should be expanded to consider slope and slope-change information, and these properties are only half of the necessary criteria; the statistical analysis of the peaks will be necessary before the information is complete. Finally, the statistical information must be correlated with subjective pilot ratings.

Nondestructive Testing of Airport Pavements

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A study of nondestructive testing techniques for the evaluation of airport pavements is summarized. The report includes (a) the selection and preparation of specifications for nondestructive testing of airport pavement systems, (b) the development of a methodology for evaluating the load-carrying capacity of airport pavement systems by using the equipment selected, (c) the development of an evaluation procedure based on this methodology, and (d) the development of a mathematical model that describes pavement response to dynamic loading.

The current methods for evaluating the load-carrying capacity of airport pavements require direct sampling techniques that are both costly and time-consuming. Often, these methods require the closing of the pavement facility to traffic operations, which necessitates the rerouting or rescheduling or both of aircraft. With the number of traffic operations increasing rapidly, even the brief closing of a pavement facility can result in inconvenience to the traveler and higher costs to the air carrier. Also, the increasing gross masses and increasing numbers of operations of aircraft make the need for accurate and frequent evaluations of pavement systems extremely important to the airport owner because many facilities will need strengthening or rehabilitation to meet these increased demands. Given these considerations, the need for a procedure that permits rapid evaluation with a minimum of disturbance to normal traffic operations is evident. The use of nondestructive testing techniques to determine the pertinent characteristics of pavement systems offers the best promise of serving this need.

PURPOSES

The purposes of this study were to

1. Select and prepare specifications for equipment to be used for nondestructive testing of airport pavement systems,

Figure 1. DSM curve and calculation.

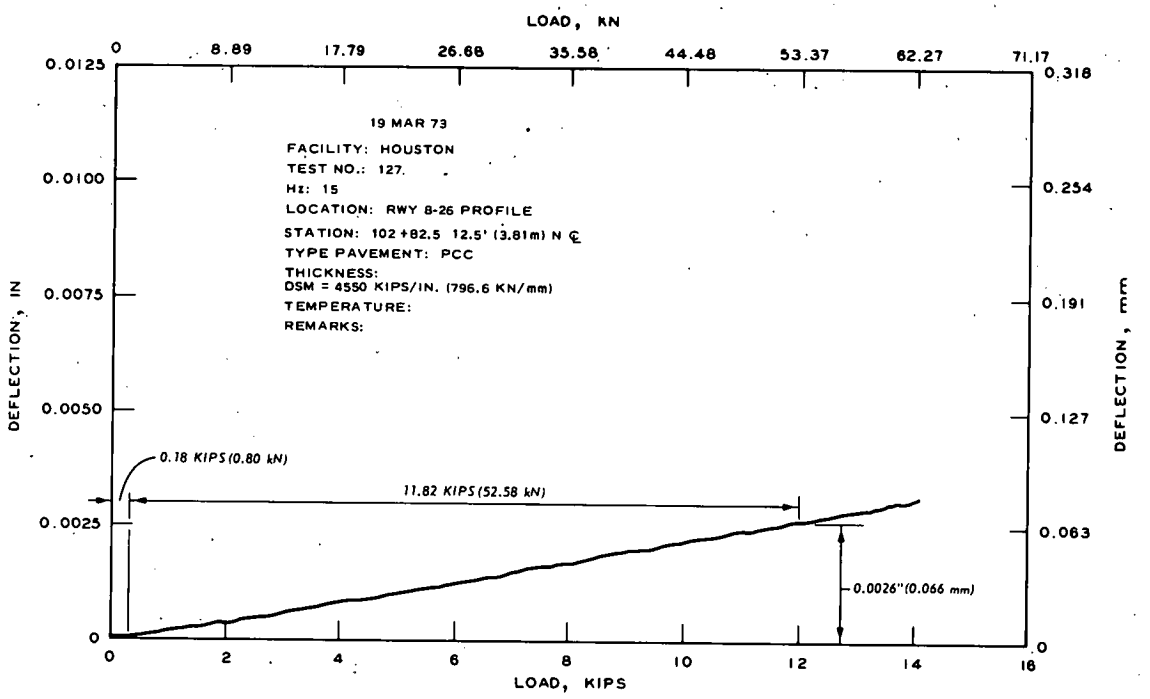
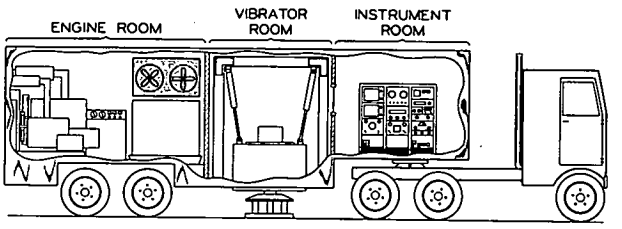


Figure 2. WES nondestructive testing equipment.



2. Develop a methodology for evaluating the load-carrying capacity of airport pavement systems by using the selected equipment,
3. Develop an evaluation procedure based on this methodology, and
4. Develop a mathematical model that describes pavement response to dynamic loading.

SCOPE

Various types of nondestructive testing equipment were studied through comparison tests on a wide range of pavement systems. The features evaluated included vibrator static masses, peak vibratory loads, methods of application of the vibratory load to the pavement (types and sizes of load plates), frequencies of loading, and mobility and ease of operation.

The tests were performed at nine airports and on test sections at the U.S. Army Engineer Waterways Experiment Station (WES). The data collected by nondestructive methods included dynamic stiffness modulus (DSM) values, deflections for frequency sweeps from 5 to 100 Hz, deflection-basin measurements, and wave propagation. (The DSM is the inverse of the slope of a load-versus-deflection curve. An example curve and calculation is shown in Figure 1.) The data collected by the direct sampling tests included the thicknesses of all layers of the materials composing the pavement sections, foundation strength values [California bearing ratios or modulus of subgrade reaction (k) values], concrete flexural strengths, and material classifications. The environmental factors considered included temperature and frost effects. Both rigid and flexible pavements were studied.

SUMMARY OF WORK ACCOMPLISHED

Selection of Equipment

Five vibratory testing devices were evaluated: the WES 71-kN (16 000-lbf) vibrator (Figure 2), the Civil Engineering Research Facility nondestructive pavement test van, the WES 40-kN (9000-lbf) vibrator, the Dynaflect, and the model 505 Road Rater. The results of the comparison tests were plotted in the form of DSMs obtained with the WES 71-kN vibrator versus DSMs obtained with each of the others, and the standard error of estimate was computed for each plot.

The effects of the vibrator static mass on the load-deflection measurements were studied by using a WES variable static-mass 222-kN (50 000-lbf) vibrator. Data taken on four different pavement items showed that the DSM increased significantly as the static mass applied to the pavement increased.

The effects of the vibrator dynamic load on the load-deflection measurements were studied with the WES 71-kN vibrator. Generally, the load-versus-deflection plots obtained with this equipment tended to curve at the lower force levels and become linear at the higher force levels [44.5 to 67 kN (10 000 to 15 000 lbf)], especially on weak flexible pavements.

The tests with the 71-kN vibrator also showed that the deflection responses of rigid and flexible pavements varied appreciably with changes in frequency. Earlier studies by WES had indicated that 15 Hz is the optimum frequency for deflection tests because the variations of the stress and deflection measurements with depth are greater at 15 Hz than at any other frequency within the capability of the vibrator.

Tests with the WES 71-kN vibrator using 30.5-, 45.7-, and 76-cm (12-, 18-, and 30-in) diameter load plates on

flexible pavement test sections showed that the effect of changes in load-plate diameter on DSM values is significant.

Accuracy and reproducibility tests with the WES 71-kN vibrator indicated that it is a reliable measuring device.

Development of Methodology

Tests for the development of the evaluation methodology were conducted at the following facilities: (a) National Aviation Facilities Experimental Center, Atlantic City, New Jersey; (b) Houston Intercontinental Airport; (c) Baltimore-Washington International Airport; (d) Greater Wilmington Airport, Wilmington, Delaware; (e) Philadelphia International Airport; (f) Jackson Municipal Airport, Jackson, Mississippi; (g) Nashville Metropolitan Airport; (h) Weir-Cook Municipal Airport, Indianapolis; and (i) WES. The pavements were characterized by direct-sampling techniques and conventional testing methods after the nondestructive tests had been performed.

The effects of temperature on DSM measurements of flexible pavements were observed on a temperature test section constructed at WES, and temperature-adjustment-factor curves that allow adjustment of the DSM values to a constant mean temperature of 21°C (70°F) were developed.

The effects of freeze-thaw cycles on DSM measurements were observed at Truax Field in Madison, Wisconsin. Core holes were drilled at test sites to determine pavement thicknesses and pavement and subgrade materials, and a graph of DSM values versus time was developed that showed an increase in deflection after the beginning of thaw. However, the development of a correction factor to reduce loads as determined by evaluation when pavements are not affected by thaw was hampered by the complicated testing conditions and the poor drainage properties of the subgrade materials.

The tests on rigid pavements showed that their DSM values could vary significantly from slab edges to centers. Therefore, the DSM tests were performed at slab centers to obtain consistent results. Variations in DSM values on trafficked and untrafficked areas of flexible pavements indicated that tests on these pavements also should be carefully located to accurately reflect the condition of the area to be evaluated.

The basic elements of the nondestructive evaluation methodology are described below:

1. Flexible pavements: The nondestructive evaluation procedure for flexible pavements uses a measurement of the rigidity of the total pavement system (the DSM) and does not consider the individual parameters that affect pavement response. The methodology is based on establishing correlations between the DSM and an allowable single-wheel load. These correlations were developed for a single wheel having a tire-contact area of 0.164 m^2 (254 in^2), 1200 annual aircraft departures, and a 20-year life. After the DSM versus allowable-single-wheel-load relation was developed, the methodology was based on existing interrelations among pavement thickness, load, load repetitions, soil strength, and landing-gear characteristics.

2. Rigid pavements: As for flexible pavements, the nondestructive evaluation procedure for rigid pavements uses a measurement of the overall rigidity of the total pavement system (the DSM). The DSM of a rigid pavement is a function of the pavement thickness and the concrete and foundation load-deformation characteristics. Again, the methodology is based on a correlation of DSM with an allowable single-wheel load. The relation between the single-wheel load and gears of different geometries is based on the equivalency of maximum bending stress in the concrete slab.

Nondestructive Evaluation Procedure

Determinations of allowable multiple-wheel aircraft loads using DSM values require that the pavement thickness (t) be known for flexible pavements and that a foundation-strength factor (F_F) and the pavement thickness (h) be known for rigid pavements. These parameters can be determined from the information given in construction drawings or from tests in small core holes. Only one core hole is necessary for determining the parameter for a feature that has uniform properties. Pavements that contain chemically or bituminous-stabilized layers can be evaluated by using equivalency factors that convert the thicknesses of stabilized pavements to thicknesses of conventional pavement sections.

Because of the ease with which the measurements are made, at least 30 DSM values should be determined in each paved area. A representative DSM value for each area is the arithmetic mean of the DSMs measured for the area minus one standard deviation.

A correction for the effects of freeze-thaw cycles on DSM data has not been developed, and so DSM measurements should not be made when pavements are frozen. A correction for temperature effects on flexible pavements, however, has been developed. Mean pavement temperatures for use in calculating the correction can be determined by measuring the pavement temperatures at points 2.5 cm (1 in) below the surface, 2.5 cm above the bottom of the surfacing layer, and midway between and taking the mean of the three readings.

The data used in developing the correlations of DSM and gross aircraft mass were collected from areas of pavements that were free of surface defects. Therefore, data collected for evaluation purposes should also be from areas in good condition. For a pavement where defects exist, the allowable load should be reduced by a judgment factor.

Mathematical Model

A mathematical model was developed based on nonlinear oscillator theory of the response of pavements subjected to dynamic loading. In this model, the dynamic stiffness is described by the following characteristics: dynamic load, static load, frequency, load-plate radius, and elastic modulus of each pavement layer. The elastic restoring force of the pavement is described by linear, cubic, and fifth-order terms in the displacement of the surface, so that three basic parameters are required by the model. These three parameters are determined from the dynamic load-deflection characteristics. The model uses a finite depth of influence that depends on the static load and the area of contact of the vibrator load plate with the pavement surface. The nonlinear behavior of the dynamic load-deflection curves is due to the finite depth of influence. With increasing load-plate sizes, the depth of influence increases and passes through the successive pavement layers.

SUMMARY OF FINDINGS AND CONCLUSIONS

A nondestructive evaluation procedure for flexible and rigid airport pavements and equipment specifications for a nondestructive testing device were developed. It was concluded that additional studies are warranted of vibrators, deflection measurements on composite pavements, theoretical relations between vibrator data and allowable loads, relations between deflection data and the proper-

ties of rigid pavement slabs, relations between deflection data and temperature, wave-velocity techniques for the determination of elastic properties, relations between deflection data and pavement performance, and effects of pavement overlays on vibrator results.

The mathematical model describing the nonlinear response of pavements can be used to predict the dynamic stiffness of a pavement given the loading conditions on the pavement directly under an aircraft wheel, to correlate the different values of dynamic stiffness measured by different vibrators at the same pavement location, and to predict the thickness and elastic moduli of each pavement layer in terms of the measured values of the dynamic stiffness for a series of load-plate sizes.

ACKNOWLEDGMENT

This paper is a summary of material prepared for the Federal Aviation Administration (1, 2, 3).

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Abridgment

Nondestructive Testing: Frequency Sweep

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A nondestructive testing (NDT) system was used to determine the physical conditions of existing pavement systems. The characteristics of the test were a static load of 71 kN (16 000 lbf) superimposed with a constant sinusoidal dynamic load and frequencies ranging from 5 to 80 Hz (the frequency sweep). For testing, a dynamic load of 17.8 to 53.4 kN (4000 to 12 000 lbf) could be selected. Each such test requires approximately 10 min and costs approximately $30 (a conventional plate-bearing test requires approximately 1.5 d and costs approximately $1500). The nondestructive nature and the rapidity of NDT minimize interference by the testing to aircraft operations, provide a better indication of the variations in the pavement support condition, and reduce the cost of testing.

The NDT procedures were standardized, the data were analyzed, and the results were used in pavement evaluation and functional design. Three subsystems in a computer system are used for the analysis, evaluation, and design. In the first subsystem, aircraft response is related to pavement smoothness, and the capacity of a pavement to withstand repeated aircraft loading is related to the user's requirements and demand forecast and to the need for maintenance. In the second subsystem, the required pavement thicknesses and composition are determined that meet the current and future requirements. In the third subsystem, the cost/benefit aspects of alternative pavement designs are evaluated to provide airport operators with realistic criteria for planning future pavement needs.

Before final adoption of the entire system of frequency-sweep NDT and its associated pavement evaluation and functional design procedures, a recommendation is made to conduct a validation program at four airports.

Abridgment

Nondestructive Testing of Flexible Pavements by Using Prototype Loads

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A major problem confronting persons concerned with the operation of airport pavements is that of evaluating existing systems. The requirements imposed by the rapid advance of air transportation and aircraft developments have outstripped evaluation techniques borrowed from highway engineers. The policy of closing traffic lanes to permit repairs on highways may have significant consequences when transferred to even a single runway of a major airport. The rate and magnitude of loadings imposed on airport pavements today have markedly increased failures and the consequent closing of runways (1) for repairs.

Technology must provide the hardware and methodology for evaluating existing pavements, for forecasting future situations and requirements, and for directing remedial measures. Operational restrictions require that procedures be developed that reduce to a minimum the closing of runways and their appurtenances, which thus precludes the use of destructive testing techniques such as test pits. Of additional importance is that destructive techniques are necessarily confined to relatively small areas of pavements, so that at best, they can provide diagnostics of only limited sample points and these at considerable cost and time.

The volume changes that can occur in response to ambient conditions can cause pavement surfaces to curl and warp. Hence, portions of the surface may not be in contact with the underlying material when subjected to im-

Figure 1. LED beam.

Figure 2. Mobile deflection measuring device.

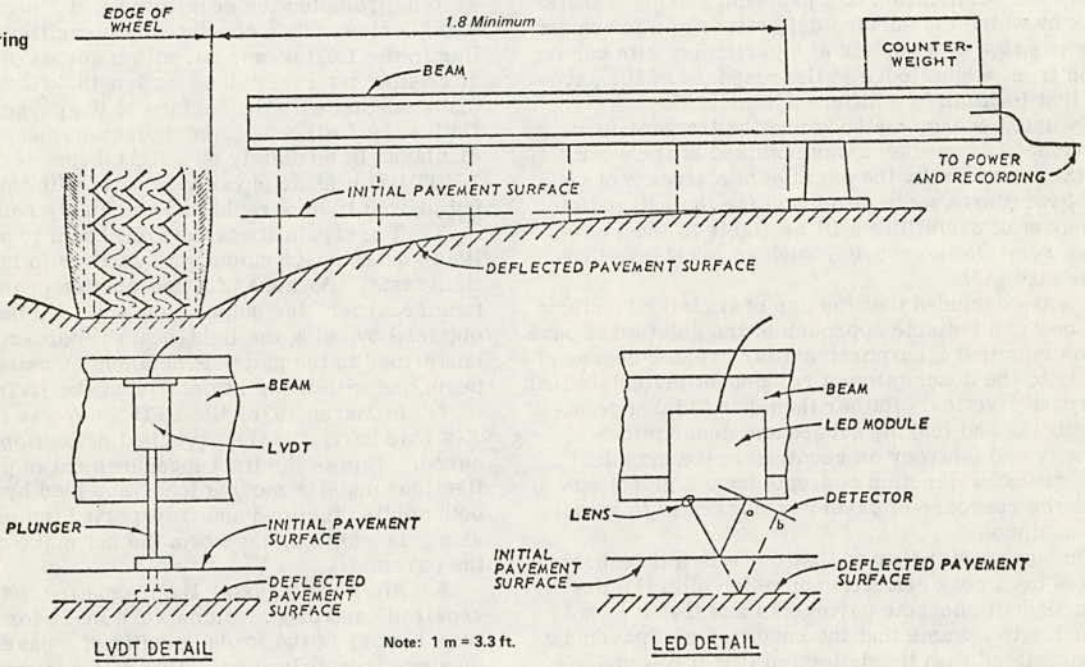

Figure 3. LVDT beam and P-2 fire truck.

posed aircraft loadings. Consequently, any apparatus used to evaluate a pavement system should not alter the conditions prevailing before loading. The devices in use today, such as the Benkelman beam and vibrators, all suffer from this limitation. Briefly, the test procedure for the Benkelman beam involves positioning the beam next to a stationary load vehicle and then measuring the rebound of the pavement as the vehicle moves away. Vibrators, of necessity, must seat the pavement before introducing safety-state vibrations, and the nature of their loading (magnitude and frequency) has little resemblance to the transient input of actual aircraft. Although Benkelman beams treat vehicular loads, they monitor only residual deflections after the pavement surface has been seated and for vehicles at creep speeds.

Needless to say, to gain widespread acceptance and use, new hardware must be (a) inexpensive; (b) operable with minimal (or no) training on the part of the user; (c) lightweight, self-contained, and mobile; and (d) able to accommodate the vehicles (or aircraft) on hand at facilities.

DISCUSSION OF RESEARCH

In November 1970, a research project at the School of Civil Engineering at Purdue University was begun that sought to exploit the concept of transfer functions to predict pavement deflections caused by moving aircraft, and in October 1971, it was extended to the examination of the time-dependent relations between the energy imparted to a pavement by aircraft and the condition of the pavement system. The field investigations were conducted at Kirtland Air Force Base, Albuquerque. Time-dependent deflection-response functions were measured with specially designed deflection gauges using linear variable differential transformer (LVDT) displacement transducers. The overall system was designed to simultaneously monitor the vertical movement of a pavement at six different locations on a line normal to an aircraft's path. The prototype investigations were conducted at three different cross sections. Four prime movers were used at each test site—the C-135, C-131, and C-130 military aircraft and a specially designed (F-4) load cart. The following results were obtained:

1. The state of the art was advanced by the development and field verification of a procedure using transfer functions by which the surface deflections of a pavement after the passage of a vehicle at a particular site can be predicted from a knowledge of the response of the pavement at that location to a different vehicle (2).
2. By using energy methods and the transfer functions developed, a measure was obtained of the work done to the pavement by the passage of a stream of vehicles. Procedures were developed for the estimation of the amount of useful life still available to the system and of the remedial measures, such as overlays, that should be taken (3).
3. It was concluded that the use of transfer functions offers a new and reliable approach to the solution of pavement problems that incorporates a far greater degree of versatility to the description of component materials than does current practice. Rather than determining pointwise attributes and making subsequent assumptions of homogeneity and isotropy as required in the classical theories, transfer-function concepts have a global approach to the response of pavement materials to actual aircraft loadings.
4. The analyses indicated that there is a threshold cumulative total peak deflection at which distress develops in asphalt concrete pavements and that it is not unreasonable to assume that the condition of a pavement can be correlated with the deflection that it has undergone. The field testing provided insight into the effect of the thickness of overlays on the energy imparted to a pavement and has led to the development of a procedure for the determination of the thickness of an overlay so that the pavement will perform satisfactorily under an anticipated traffic volume.

After the concepts of transfer functions and an energy-related distress criterion had been verified at the three LVDT installations, the next phase of the research was directed toward developing a breadboard model on a mobile pavement-deflection measuring system. This type of system obviates the need for the installation of difficult and expensive built-in LVDTs. In addition, it extends the evaluation to an entire runway-taxiway (global) system from the single transverse line of embedded gauges (local). The concept of the measuring system was to move the length of the pavement and in transit compile the inherent transfer functions and their characteristics. A secondary object was to provide the capability of rapidly defining the time history of the loaded pavement under moving aircraft. The development of the equipment for measuring the pavement deflection was based on a study of runway-deformation contouring by using holographic techniques. The development of the mobile device was based on the observation that a point on the surface of a pavement 1.8 m (6 ft) lateral from the edge of an aircraft wheel showed no measurable deflection. Consequently, attention was directed toward the construction of a cantilever beam that would span this distance and provide a horizontal datum from which vertical deflections could be measured. The following procedures were used:

1. A beam was constructed that uses light-emitting diodes (LED) to reflect coherent light from the pavement onto detectors (Figure 1). Then as the pavement deflects, the reflected light will register differently on the detector (as shown in Figure 2). The calibration of the beam provides a scale that relates the differences obtained on the detector to the vertical displacements. No physical contact is required between the beam and the pavement at points of measurement. The data were stored on a magnetic tape recording system.
2. Early difficulties with the electronics of the LED system prompted the development of a secondary (fail-safe) device. This consisted of a cantilever beam similar to the LED beam, but with a series of LVDTs spaced at regular intervals along its length, whose plungers made contact with the surface of the pavement (Figure 3). Unlike the LED beam, the deflection measurements were displayed immediately on a light-beam recorder. The LVDT beam made physical contact with the pavement, but proved to be a rapid and extremely reliable device.
3. The capability of the LED beam to provide reliable quantitative output was verified in laboratory and field tests. Because of a malfunction of the magnetic tape recorder, the output from the LED beam had to be obtained by using the light-beam recorder. The results conformed to the patterns obtained by using the LVDT beam and with those registered at the LVDT gauges.
4. In March 1976, the LED beam was mounted on an F-4 load cart, and the pavement deflections were measured. This is the first measurement of pavement deflections under a moving load, obtained by a vehicle that both applied the load and transported the measuring device. In addition, the beam did not make contact with the pavement.
5. After these tests, the magnetic tape recorder was repaired, and preparations were made for obtaining the time history of the loaded profile of a pavement under a moving aircraft loading. This was accomplished in June 1976. The LED beam was mounted on the F-4 load cart, and deflection measurements were obtained for more than 605 m (2000 ft) of taxiway and 2000 m (1.25 miles) of runway—all in less than 3 h (including the time taken because of damage to a gauge that then required extra calibrations).

SUMMARY AND CONCLUSION

A breadboard model of a nondestructive, noncontact, rapid, mobile, global device for measuring the response of pavements to moving aircraft has been developed and tested under field conditions.

ACKNOWLEDGMENT

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structed by Science Applications, Incorporated, Albuquerque.

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Abridgment

Pavement Evaluation With the Falling-Weight Deflectometer

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The falling-weight deflectometer (FWD) is a reliable, simple, and yet effective tool to determine the structural properties of pavements for roads and runways. Its characteristics of level of force (up to 60 kN) and loading time (approximately 30 ms) are more representative of heavy traffic than are those of most other deflection equipment. The deflection of the pavement and the deflection bowl are determined by geophones (velocity transducers) in the center of the loaded area and at certain distances from the center. The deflection levels are not affected adversely by the configuration of loading or the recording system, nor by possible influences of unstable reference points such as are encountered with some alternative systems.

A comprehensive description of the method has been published elsewhere (1). The principles are as follows:

The pavement structure is represented by a multilayer linear elastic system in which the materials are homogeneous and isotropic and are characterized by Young's modulus of elasticity (E) and Poisson's ratio (ν). In most cases, the structure can be regarded as a three-layer system (Figure 1) consisting of a bound top layer (E_1, ν_1, and thickness h_1), an unbound or cemented base layer (E_2, ν_2, and h_2), and a subgrade (E_3, ν_3, and infinite thickness). To simplify the system, fixed values are adopted for ν for all layers. The test load is assumed to be uniformly distributed over one or more circular areas.

The response of the pavement to a test load is characterized by the maximum deflection and the shape of the deflection bowl. The latter parameter is characterized by the ratio (Q_r) of the deflection at a distance r from the load (δ_r) to the deflection under the center of the test load (δ_0); i.e., $Q_r = \delta_r/\delta_0$. The ratio Q_r was chosen rather than the radius of curvature because Q_r can be measured more easily with existing equipment and provides equivalent information. The distance r can be fixed depending on the type of structure and is preferably such that Q_r is approximately 0.5 (2).

By using the BISAR program, graphs have been prepared giving the relations among E_1, δ_0, Q_r, and h_1 for predetermined values of E_2, h_2, E_3, and r for a given test load. From such graphs, with δ_0 and Q_r measured, two unknown structural parameters of the pavement can be determined if the other variables are known or can be estimated.

The pulse load is applied by a mass falling onto a set of springs that are mounted on a rigid circular plate resting on the pavement surface. Extensive tests with a prototype have shown the suitability of the method for determining the shape of the deflection bowl (1).

For routine measurements, a commercially available FWD was used. This FWD is mounted on a small trailer that can be towed by an automobile. The falling mass weighs 150 kg, and the maximum force developed is 60 kN at a drop height of 400 mm and a pulse width of 28 ms. These values were almost independent of type of pavement structure. The diameter of the foot plate is 300 mm.

The deflection of the pavement is measured by velocity transducers (geophones), one in the center of the loaded area and one or two at a fixed distance from the load. The transducers (50 mm in diameter and 55 mm in height) operate over a frequency range of 1 to 300 Hz. They are very rugged, which is advantageous in field conditions.

The FWD and the trailer were modified to permit quick operation with remote control from the towing car to enable measurements to be made in or between the wheel tracks without changing the line of drive. The trailer accommodates the power supply, and the recording instruments are in the automobile.

By using this equipment, surface deflections can be measured accurately and easily without the problems (such as the measuring reference points being located in the deflection bowl) that are encountered with some alternatives. The FWD is shown in Figure 2.

The transducer signals used to record the deflections are displayed on a screen to permit the operator to quickly verify the correct operation of the equipment and instruments. The maximum values of the deflections are digitized and stored on a tape recorder via a data unit or can be tabulated by a printer. The location of the points at which deflections are measured is recorded both longitudinally and transversely (in or between the wheel tracks or other position) with respect to the geometry of the road. A desk calculator facilitates the introduction of temperature data and is also used to calculate deflection ratios, mean values, variances, and other statistical data. A chart recorder is used to display the readings and calculated results.

Most pavement structures exhibit nonlinear behavior, particularly at low force levels, but generally, at force levels >30 kN, there is a straight line relation between deflection and force (Figure 3) (2). Thus, extrapolation of the deflection values to very heavy loads such as those under aircraft can be made with reasonable accuracy from measurements at, for example, 30 and 60 kN.

The residual life is the difference between the original design life of the pavement and the life already used. Because the initial decrease in asphalt modulus is relatively small, the design life of a pavement can be determined from the layer thicknesses and moduli derived from deflection measurements by using a Shell design chart (such as Figure 4) for the relevant subgrade modulus, weighted mean annual air temperature, and type of asphalt mix (3) (the same type of mix, of course, as anticipated with the interpretation) and deriving the number of standard axle-load applications for the given values of h_1 and h_2. Depending on the difference between the de-

Figure 1. Schematic representation of pavement structure under test load.

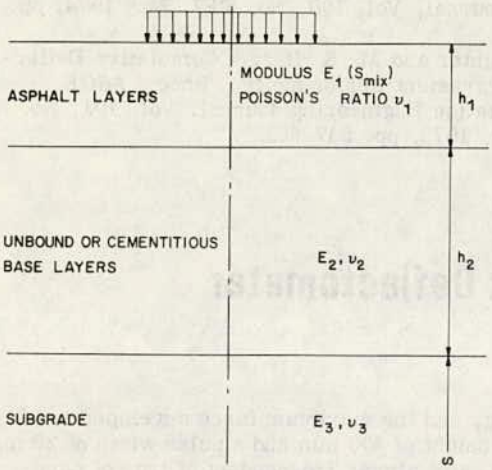


Figure 2. Falling-weight deflector.

sign life found and the traffic carried to date, it can be decided whether or not strengthening is required.

The Shell design charts can also be used to determine the overlay thicknesses required for estimated future load applications. However, in many cases, the maximum asphalt strain occurs in the underside of the asphalt layer; thus after the strengthening of the pavement with an additional asphalt layer, strain will still be present in the original layer, although at a lower magnitude. In an overlay thickness design, this reduction in fatigue life of the original asphalt mix must be taken into account. When the asphalt mix used for the overlay is essentially the same as that of the original asphalt, the overlay thickness required for the design number (N_{D2}) can be derived directly from the design chart used to determine the present design life (N_{D1}), i.e., Figure 4.

The pavement evaluation and overlay design method described is regarded as a complete method that provides the engineer with structural pavement properties

Figure 3. Deflection as function of load.

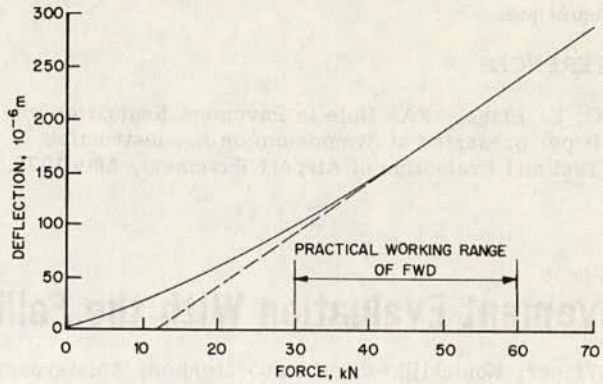
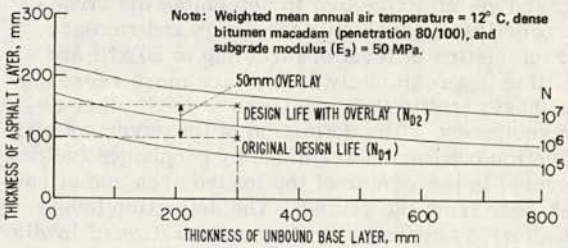


Figure 4. Use of Shell design chart to derive overlay thickness.



that can be used directly in a structural design method in which all relevant aspects (e.g., loading, climate, and use of materials) are incorporated.

The development of new design charts to replace the Shell design charts for flexible pavements published in 1963 and the pavement evaluation and overlay design procedures outlined above are described in detail elsewhere (3, 4).

Evaluation with the FWD has been used effectively on many roads and at three major airports (Zurich, Amsterdam, and Brussels) in Europe.

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Comparison and Evaluation of Nondestructive Testing Methods of Airport Pavement Evaluation

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Four methods of nondestructive testing for airport pavements—steady-state vibratory, both load-sweep and frequency-sweep procedures, wave propagation, falling weight deflectometer, and deflection profile—are compared. Their advantages, especially in terms of costs and ease of operation, and their limitations, especially in terms of the required correlations between pavement property measured and pavement performance, are summarized. The steady-state vibratory methods are recommended, and a program for their use is suggested.

There is strong and rapidly growing interest in the development of nondestructive methods for evaluating the load-carrying capacities and potential performances of airport pavements. This interest in nondestructive testing (NDT) has been spurred by advances in equipment and technology that have made NDT a realistic goal and the need for testing procedures for use in pavement evaluation that cause a minimum of disruption to normal airport operations. The pressure for nondisruptive test procedures will increase with increasing air traffic.

To be of value to the airport pavement engineer, NDT methods must meet the following criteria:

1. They must be rapid,
2. They must be simple enough so that they do not require highly trained engineers to conduct the tests,
3. The costs of conducting the tests and interpreting the results must not be excessive, and
4. The results of the tests must be interpretable in terms of the potential performance of the pavement and the effects of pavement rehabilitation on the performance trends (Figure 1).

If the NDT methods cannot meet these criteria, they will either not be used or be of little value to airport engineers and managers.

At present, there are several NDT methods either under development or being applied on a limited basis. These include (a) steady-state vibrators, (b) wave-propagation procedures, (c) the falling weight deflectometer, and (d) measurement of the deflection basin under moving loads. The steady-state vibrators are the most highly developed in terms of application to airport pavement systems. This paper will focus on the practical implications of these test methods.

In recent years, two separate methods of testing and interpreting the results obtained by using steady-state vibrators have been developed. Both methods have been used on a number of airport pavements, and both use the same equipment for the testing, but there are variations in the test procedures. For simplicity in comparison of these two methodologies, it will be assumed that the 71-kN (16 000-lbf) vibrator developed at the U.S. Army Engineer Waterways Experiment Station (WES) (1) is used for both. However, it is possible to use other equipment, provided that it is capable of testing pavements under both an adequate range of vibratory-force amplitudes and over an adequate range of frequencies. Single-load and single-frequency equipment cannot be used.

STEADY-STATE VIBRATORY METHODS

One method of testing and evaluation that has been developed at WES (1) is to select a specific test frequency and conduct tests over a range of loads at this preselected frequency. This is referred to as the load-sweep method of NDT. The other method of testing and evaluation, developed by Yang (2, 3), is to preselect a load range and test the pavement by using a range of frequencies. This is referred to as the frequency-sweep method of NDT. Because the two methods of testing produce different outputs, it follows that the interpretations of the findings will also be different. The two methods are summarized below, and the implications of each are discussed.

The basic concept of the load-sweep approach is that there exists a correlation between pavement deflection and performance as indicated in Figure 2. Thus, if the deflection of a pavement under dynamic load is determined, the resistance to deflection can be expressed as a dynamic stiffness modulus (DSM) as follows:

$$DSM = F/\delta \quad (1)$$

where

F = amplitude of the dynamic force applied at the preselected frequency (usually 15 Hz) and
δ = deflection of the dynamically loaded pavement.

The steps involved in the procedure are summarized below:

1. Conduct tests over a range of dynamic loads at a frequency of 15 Hz and a sufficient number of locations to identify the critical pavement segments,
2. Determine the DSM values for the critical pavement segments,
3. Determine the pavement capacity by correlating the DSM values with previously established curves relating DSM to capacity for various types of pavements,
4. Establish total pavement section requirements for proposed traffic conditions, and
5. Establish pavement rehabilitation and overlay needs from the differences between the answers to steps 3 and 4.

If the pavement and subgrade systems were linear with respect to F and δ, it would be necessary to test the pavement under a fixed force amplitude. However, most pavement and subgrade systems are not linear, but have load-versus-deflection plots, such as that shown in Figure 3. The DSM value for the pavement is determined by taking the inverse of the slope of the linear portion of the load-deflection curve as illustrated in Figure 3.

The DSM is a measure of the resistance of the pavement-subgrade system to deflection under dynamic loads. In itself, the DSM is of little value in predicting pavement performance. It becomes significant only when the values for typical pavement sections are correlated (Figure 4) with allowable load capacities determined in the conventional manner by using destructive test methods. In this approach, the accuracy of the NDT and pavement-evaluation procedure is dependent on the validity of the conventional procedure used, with some possible loss of accuracy because of the errors inherent in any correlation procedure.

One of the criticisms of the DSM as a method of pave-

Figure 1. Use of NDT for pavement management.

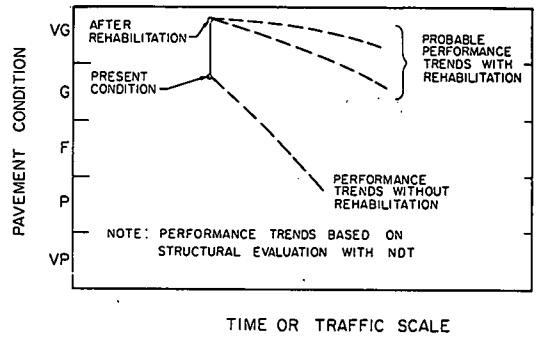


Figure 2. Correlation between pavement deflection and performance.

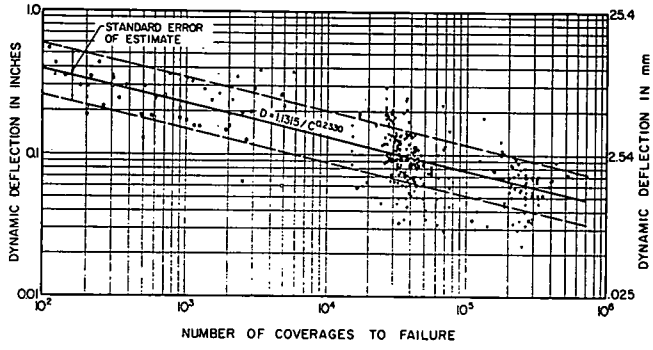


Figure 3. Deflection versus amplitude of force for typical pavement system.

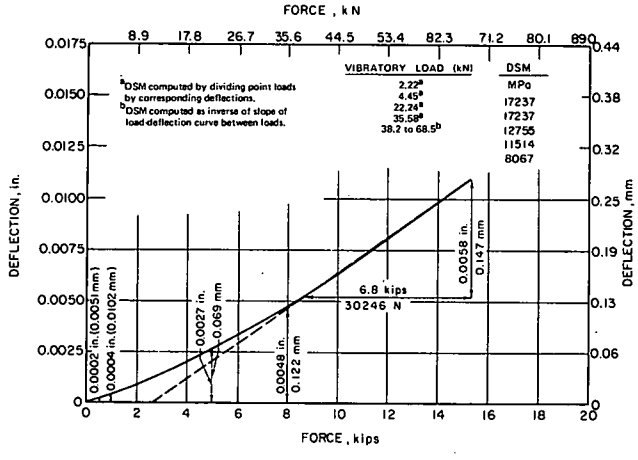
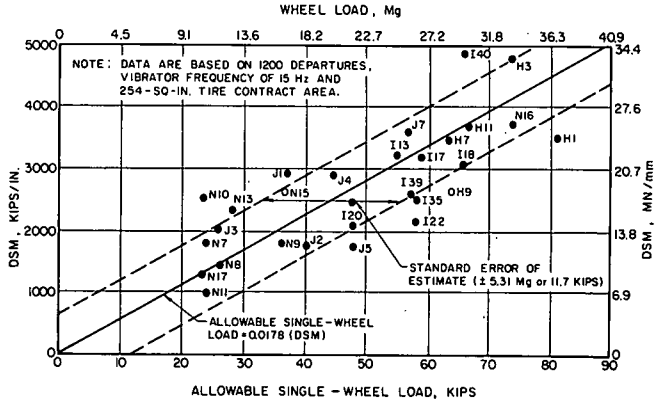


Figure 4. Correlation between DSM and allowable load as determined by conventional test and evaluation procedure.



ment evaluation is the need to preselect the testing frequency (which is normally 15 Hz). The arbitrary selection of this particular frequency is based on the relatively constant ratio observed between the pavement deflection at that value and at other frequencies for a wide range of dynamic load amplitudes as shown in Figure 5.

Different pavement systems have different natural frequencies. Figure 6 shows the effect of testing frequency on pavement deflection for a number of pavement systems at a given dynamic force. A study of these curves indicates that the first or primary mode for the resonant frequency is generally >5, but <10 Hz, although several of the pavements had an apparent resonance at a significantly higher frequency. There is considerable speculation as to the significance of the findings at test frequencies near, but not at, the first resonant frequency, but this point has not yet been fully investigated.

Because the load-sweep method of NDT relies exclusively on the correlation between the DSM and allowable loads, strict limitations must be placed on the test procedure and the methods for interpreting the results. The constraints on the method (1) are shown below:

1. The DSM must be determined by using a vibrator with a static mass of 71 kN and a load-plate diameter of 45.7 cm (18 in).
2. The DSM must be computed using the slope of the linear portion of the deflection-versus-load curve.
3. The DSM must be measured at 15 Hz.
4. A temperature adjustment must be applied to the DSM on flexible pavements.
5. The DSM for rigid pavements must be measured at the slab center.
6. The moduli of elasticity of the respective pavement layers under investigation must decrease with depth.

The basic concept of the frequency-sweep approach is

Figure 5. Effect of frequency of dynamic load on pavement deflection over range of amplitudes.

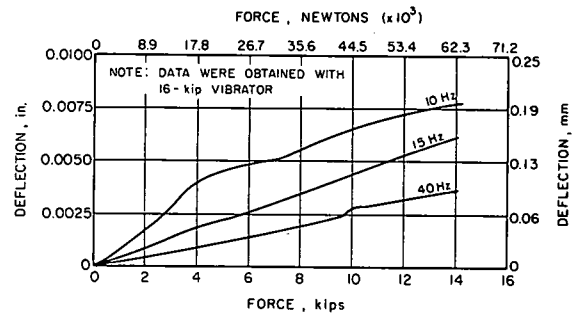
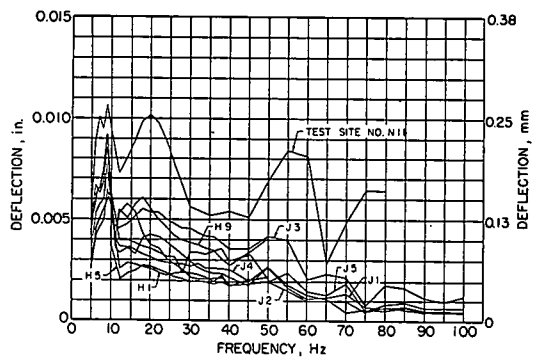


Figure 6. Effect of loading frequency on pavement deflection.



also based on the resistance of a pavement to deformation under a dynamic force. However, rather than measuring the resistance to deformation and using this value directly, Yang calculates an equivalent modulus (E) for the pavement-subgrade system. The procedure for determining E involves testing the pavement at a fixed amplitude of dynamic force over a range of frequencies from well below the first resonance to approximately 10 times the primary resonance and then using the following equation:

$$E = (F/2a) \times 1/\Sigma[\delta(u)/u] \quad (2)$$

where

F = amplitude of the dynamic force,
a = radius of the loading plate,
u = evaluation frequency as a multiple of the first resonant frequency, and
$\delta(u)$ = pavement deflection at frequency u.

The steps involved in the procedure are summarized below:

1. Test at a sufficient number of locations to define the critical pavement segments.
2. Calculate the E-values of the critical pavement sections.
3. Determine the load and traffic capacities of the existing pavement (Yang uses a combination of elastic-layer system theory and experience).
4. Establish the pavement needs for the proposed traffic.
5. Determine the pavement rehabilitation or overlay-thickness requirements for the proposed traffic.
6. Project the pavement performance and make an economic analysis of the alternatives.
7. Make engineering decisions.

Figure 7 shows the effect of frequency on pavement deflection for a given pavement. The first resonant frequency for this pavement was approximately 8 Hz. Thus, the deflection values are read at frequencies that are multiples of 8 (i.e., 8, 16, 24, 32 . . .). Each deflection is then divided by the appropriate multiple of the resonant frequency (i.e., 1, 2, 3, 4 . . .), and these ratios are summed.

Although the frequency-sweep method for quantifying the resistance to deformation of a pavement is based in part on random vibration theory, the procedure in fact relies heavily on correlations between E-values determined by NDT and E-values determined by using conventional plate-load tests on the pavement. Here two problems arise: The first is that different methods of conducting plate-load tests produce different E-values, and the second is that, because of the nonlinearity between pavement deflection and the amplitude of the dynamic force, it is necessary to arbitrarily select an amplitude of the force for the test procedure. The latter problem is illustrated in Figure 8.

Both of the procedures for NDT and evaluation discussed above have been used to successfully determine pavement load capacity and to develop realistic pavement rehabilitation plans. Both, however, have some obvious limitations and their successful application is due more to the sound engineering judgment and ingenuity of the persons involved in the evaluations and rehabilitation designs than in the fundamental correctness of the procedures. Some of the limitations, some necessary steps for improvement, and some needed research are discussed below.

The primary use of the NDT procedures is to allow the pavement engineer to predict pavement performance under specified traffic conditions, schedule necessary rehabilitation procedures, and make plans for the future. Both procedures attack this problem by measuring the resistance of the pavement-subgrade system under dynamic loads and using this information to predict pavement performance. The two procedures provide different measures of this resistance to deformation, but each provides essentially a single value to quantify this property.

Most of the deflection in pavement-subgrade systems is due to deformation in the subgrade. It has been estimated that more than 80 percent of the deflection in flexible pavements and nearly 100 percent of the deflection in rigid pavements occurs in the subgrade soil. Thus, significant improvement can be made in the NDT and evaluation procedures if some of the critical properties of the subgrade soil are known.

The left side of Figure 9 shows a representation of a typical flexible pavement system, and the right side shows a representation of what is simulated by a single-valued property of the pavement-subgrade system. In predicting the pavement deflection under a specified load, the single-valued property of the system is just as valid as is a multiple-valued designation. A problem arises, however, when one tries to evaluate the effect of an overlay or other rehabilitation with a single-valued system.

Figure 10 shows representations of a typical flexible pavement system and of its simulation by a single-valued system, both with an overlay. It is apparent that an overlay on the single-valued pavement system will have a significantly different effect on the theoretical behavior of the system than will the same overlay on the typical pavement section. Because of this deficiency, it is probable that neither NDT procedure described will provide a meaningful estimate of pavement behavior after overlay and that both in their present stage of development are not valid tools for predicting performance after rehabilitation. Fortunately, this limitation is not as difficult to correct as might be anticipated.

Because most of the deformation of a pavement is due to deformation in the subgrade, the theory for predicting pavement behavior can be improved substantially if the resistance to deformation of the subgrade is known. If the subgrade modulus is known or can be estimated to a reasonable level of accuracy, equivalent thicknesses and moduli can be calculated from elastic-layer theory (4) or the application of the Odemark theory for equivalent pavement sections (5). Also, because most of the deformation occurs in the subgrade, the equivalent thicknesses and moduli of the pavement itself need not be known to a high level of precision.

There are several ways to determine modulus values for subgrade soils. In evaluating these methods, it is well to remember that very few soils exhibit true elastic properties, but rather are highly stress-dependent and exhibit both permanent and recoverable strain characteristics. Figure 11 shows a typical load-versus-deformation curve for fine-grained soils. The ratio of the permanent deformation to the recoverable or resilient deformation depends on many factors, including the level of stress in the soil. If the stress level is high enough to cause significant permanent deformation in the subgrade soil after a number of load applications, then any type of pavement will give poor performance. Thus, most engineers rely on the recoverable or resilient deformation to predict pavement behavior.

The resilient modulus of a soil is defined as the applied deviatoric stress divided by the resilient strain in the soil specimen and is a function of the stress level (6, 7). Figure 12 shows the effect of stress level on the resilient moduli for several soils. These moduli are

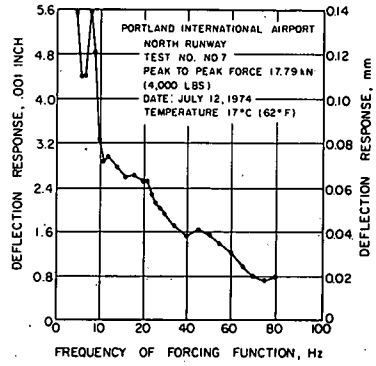


Figure 7. Relation between pavement deflection and frequency: frequency-sweep method.

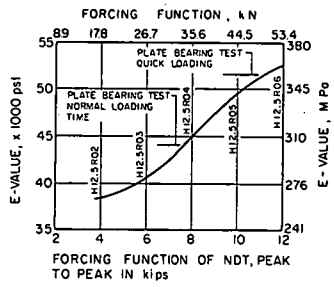


Figure 8. Effect of magnitude of dynamic force on calculated E-value: frequency-sweep method.

Figure 9. Representations of typical pavement system and single-value simulation of such a system.

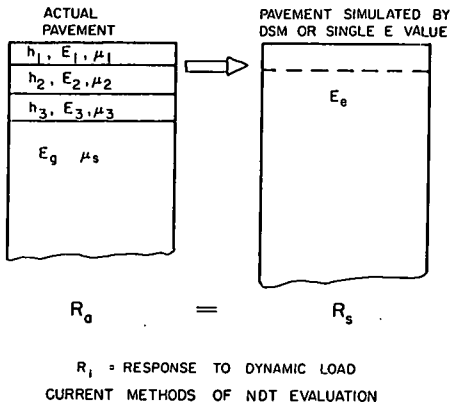
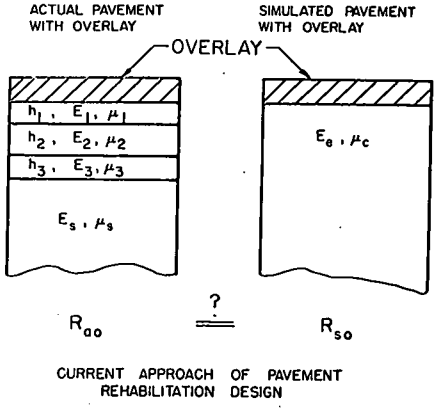


Figure 10. Representations of typical pavement system and single-valued simulation of such a system, both with overlays.



also affected by density and especially by the degree of saturation of the soil. Figure 13 shows the effect of moisture content above optimum on the resilient modulus of a typical fine-grained soil.

With all these complicating factors, it is logical to wonder how a realistic resilient modulus value could ever be established for subgrade soils. However, there have been several attempts to establish test procedures for the estimation of the resilient modulus of fine-grained

Figure 11. Typical stress deformation for fine-grained soil under impulse-type loading.

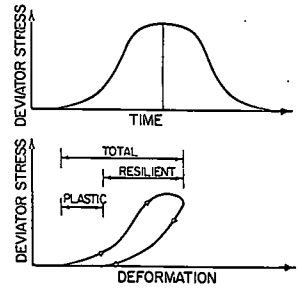


Figure 12. Effect of level of stress on resilient moduli of typical fine-grained soils over a range of strengths.

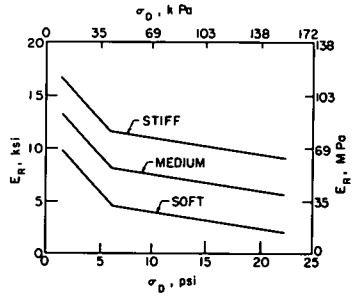


Figure 13. Effect of moisture content [in percentage relative to optimum (O)] on resilient modulus of typical fine-grained soil.

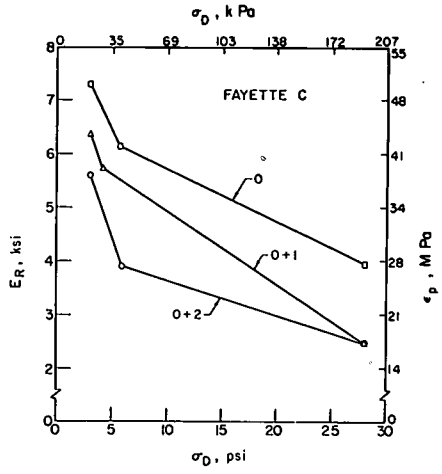


Figure 14. Stress-dependent nature of resilient moduli of fine-grained soils and typical parameters for quantification.

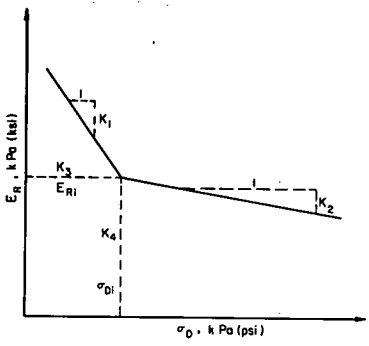


Figure 15. Wave-propagation equipment.

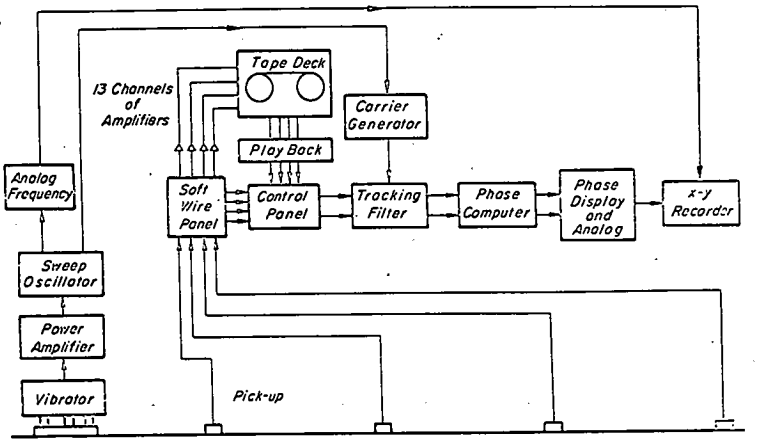


Figure 16. Typical dispersion curves from field data.

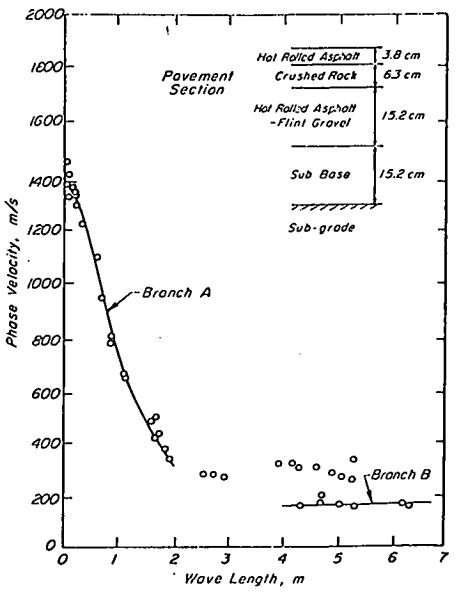


Figure 17. Relative strains and stresses in flexible pavement system caused by rolling truck wheel load and falling weight deflectometer.

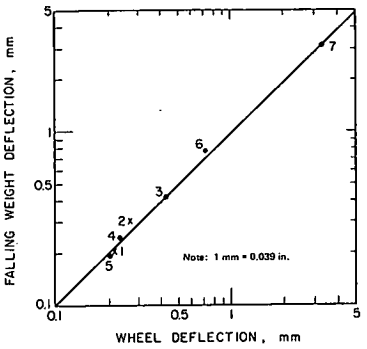
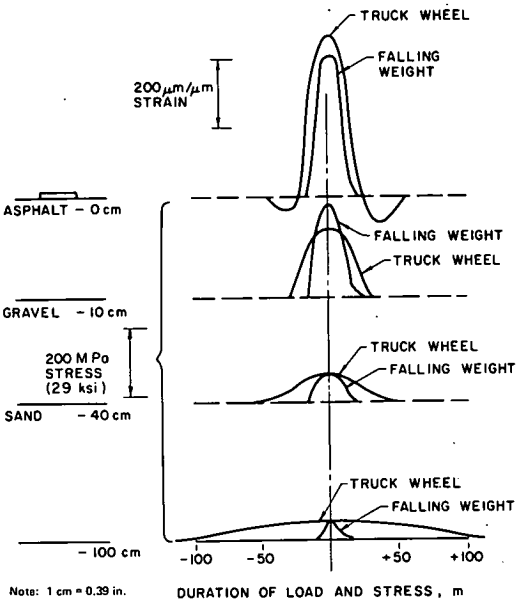


Figure 18. Comparison of pavement surface deflections produced by rolling truck wheel load and falling weight deflectometer.



soils that have excellent chances for success.

In the late 1950s, workers at the University of California, Berkeley, developed test procedures to measure resilient moduli (6) that have been refined and improved until it is now possible to measure the resilient modulus of an undisturbed soil sample quickly and accurately (7). The undisturbed soil samples for these tests can be taken through small [5.1 to 10.8-cm (2.5 to 4-in) diameter] core holes in the pavement with a tube sampler. The tube samples can then be shipped to a laboratory and quickly processed to determine the resilient modulus values for the subgrade.

If it is not possible to obtain the necessary undisturbed samples and test them, it is possible to obtain a good estimate of the resilient modulus values from key indicator values for the subgrade soil. Thompson and Robnett (7) have shown, based on tests on approximately 50 Illinois soils, that the resilient modulus of a soil can be estimated by the following equation:

$$E_{ri} = 6.37 + 0.034C + 0.45PI - 1.64\overline{OC} - 0.0038S - 0.244GI - 0.344M \quad (3)$$

where

E_{ri} = resilient modulus in kips per square inch as illustrated in Figure 14,
C = percentage of clay in soil (less than 2 microns),
PI = plasticity index of soil,
$\overline{OC}$ = percentage of organic carbon in soil,
S = percentage of silt in soil (-200 sieve to 2 microns),
GI = group index of soil (determined by AASHO Soil

Classification procedure), and
M = moisture content (percentage) above optimum.

(The operation of this equation requires that the variables be given in U.S. customary units.)

For the 50 soils tested, this equation has a correlation coefficient (R) of 0.80, with a standard error of the mean of 2.18 kips per square inch. Of all the factors listed, the resilient modulus is most sensitive to the moisture content of the soil. Thompson and Robnett (7) show, for example, that there is a high degree of correlation between E_{rt} and the degree of saturation for the 50 fine-grained soils tested, taken collectively. The more information that is known about a soil, however, the more reliable the prediction of the E_{rt}. Thus, if the type of subgrade soil is generally known, so that such factors as the clay content, silt content and PI can be estimated or if the moisture content of the soil can be determined, either by sampling or with a nuclear probe, then it will be possible to make reasonable estimates of the resilient moduli for subgrade soils with a minimum of destructive testing.

These procedures for the characterization of subgrade soil are not necessarily recommended, but are given to illustrate the current state of the art and what might be done with a little imagination and engineering judgment. A good estimate of the soil dynamic properties and some knowledge of the composition and thicknesses of the pavement layers will permit substantial advances in pavement NDT and evaluation without additional advances in technology or significantly increased costs.

WAVE-PROPAGATION METHOD

Another NDT method that has received considerable attention and research funding is wave propagation. This method has been under development since the mid-1930s, but has not yet been used to any significant degree for pavement evaluation.

The basic concept of the approach is that there is a relation between the speed of surface wave propagation and the shear modulus, density, and Poisson's ratio of the pavement material that can be expressed as follows:

$$\gamma = n(G)^{1/2}/\rho \qquad (4)$$

where

γ = propagation velocity of the Rayleigh wave,
G = shear modulus,
ρ = mass density, and
n = function that depends on Poisson's ratio.

Watkins and others (8) have shown that there are other types of surface waves generated, but the primary wave form used for this analysis is the Rayleigh wave.

The testing procedure for wave propagation is represented in Figure 15 (8, 9). A vibrator is used to excite the pavement system, and sensors that can be moved relative to the vibrating source are used to record the phase velocity and wavelength of the surface wave.

The depth to which the waves penetrate the pavement system is a function of the wavelength, which is inversely related to the frequency. That is, a low-frequency vibration will have a long wavelength, and a high-frequency vibration will have a short wavelength. Thus, by measuring the wave-propagation velocity and the wavelength, it is possible to establish the values of the shear modulus at various depths in the pavement. Dispersion curves of the types shown in Figure 16 are thus generated from the data collected. Computer programs have been developed to calculate representative shear moduli for each of the pavement layers (8).

Watkins and others (8) list the following advantages for this method:

1. The method is nondestructive.
2. The materials are tested in situ. This eliminates the need to account for a number of variables that are difficult to reproduce with precision in the laboratory (e.g., the confining pressure resulting from a complex stress field).
3. The properties of the materials are determined over a wide area, rather than from a localized test sample.
4. The properties of the materials are determined in the form of their various moduli. This allows their use in rational analyses of the response of the structure to loading.
5. There is a minimum of disruption to site activity, which is of major importance in testing highway pavements and airport runways.

There are also some substantial disadvantages of limitations on the use of wave-propagation methods. These are summarized by Watkins and others as follows:

1. The stress levels induced in the layered structure are generally much lower than those produced by working loads. As many natural materials have nonlinear stress-versus-strain characteristics, moduli determined by testing cannot be used directly in the analyses of structure under traffic loading, but must be modified to incorporate the effects of stress level.
2. The equipment is relatively expensive.
3. The complexity of the equipment and the need for a rather sophisticated interpretation of the data require an operator with some knowledge of electronics, an understanding of the nature of wave propagation in layered media, and considerable experience with the technique. This may reduce the number of routine applications for which the method is useful.
4. The interpretation of the data is based on the ability of theoretical analyses to correctly predict the nature of wave propagation in layered media. At present, these methods are insufficiently advanced to allow full advantage to be taken of data available from testing programs. Reliable information is available for only the simplest of structures, such as a single stiff layer over a soft half-space, and in many cases, simplifications and idealizations are used whose practical implications are not fully understood.

This last limitation is probably the greatest single obstacle to the widespread use of the wave-propagation method. Its solution requires advances in the quality of the theoretical models used to represent practical structures and the full exploitation of existing analytic tools. The foundations of these analytic methods are based on a mathematical representation of the basic phenomena of wave propagation.

FALLING-WEIGHT DEFLECTOMETER METHOD

A recent advance in NDT has been the development of the falling-weight deflectometer (FWD) (10, 11). The test equipment and procedure includes a mass of 150 kg (330 lb) that is lifted to a predetermined height [usually 40 cm (15.7 in)], and released. The mass falls free, guided by a vertical rod, and impacts on a spring-shock system resting on the pavement. The magnitude of force and duration of impulse on the pavement can be controlled by changing the height of drop and the rate of decelera-

tion (spring-shock system).

A 76-cm (30-in) diameter plate transmits the load caused by the decelerating mass to the pavement, and the deflection of the pavement is recorded. Figure 17 shows the magnitude and duration of force transmitted to the pavement in a typical test. Figure 18 shows a correlation between pavement deflections under a conventional truck wheel load and the FWD (11).

The data obtained with the FWD is essentially the same as that obtained by using a Benkelman beam and a loaded truck; i.e., the value of a deflection under a specified load. These data are useful in projecting potential pavement performance only insofar as the pavement deflection can be correlated with pavement performance. All the comments made concerning the steady-state vibratory test methods and pavement performance apply equally to the FWD method of testing and evaluation.

There are several advantages of the FWD method of NDT over the steady-state vibratory method. These include that the equipment is inexpensive compared to the waterways experiment station vibrator and that it is relatively small and so can be transported by truck, rail, or aircraft. Another advantage is that the type of load applied to the pavement is more compatible with the load pattern applied by moving aircraft gear, especially if the aircraft is moving at a significant speed.

The major disadvantage of this type of NDT procedure is that its results can be interpreted only through correlations of pavement behavior and performance, and such correlations do not always lead to precise evaluations of pavement performance. At present, there are very little background data to use in the correlation process, so that this method cannot be used with confidence. Also, there is little or no theoretical basis to use in developing the necessary deflection-performance relations other than that obtained from the study-state vibratory tests.

DEFLECTION-PROFILE METHOD

Another method of NDT is that proposed by Harr and discussed in a paper in this Special Report in which the pavement deflection and the deflection profile, called a signature, are continuously measured by a moving reference beam attached to a moving vehicle that, in turn, loads the pavement with the moving load. Because the pavement deflection is measured by light sources and sensors, no part of the pavement-deflection measuring system actually touches the pavement. Thus, the system can be moved along the pavement while recording the deflections under the moving wheel load.

While this system may have the potential to be developed into a useful procedure, it has not yet been used in an actual evaluation of any pavement system other than experimentally. Also, because the evaluation is based solely on pavement deflection and pavement-deflection profiles, the results can be related to pavement-performance evaluation only through a correlation of deflections and performance. Thus, the procedure is subject to the same limitations and correlation problems as the vibratory and FWD procedures. Also, the correlation procedures for the moving deflectometer may be the same as those of the other NDT procedures, but the actual correlations may be different because of the different loading procedures.

CONCLUSIONS AND COMMENTS

Practical NDT procedures have been developed for the evaluation of airport pavements. At least two of these procedures have been used for the evaluation of civil airports in operation and their results used to develop practical pavement rehabilitation plans. Because NDT procedures permit pavement evaluation with a minimum disruption to normal airport operations, their use will probably be greatly expanded as air traffic increases at major airports.

While the current NDT procedures have been effective in providing realistic pavement evaluation and rehabilitation plans, this has been due in large measure to the experience and good judgment of those engineers who have been associated with the testing and evaluation programs. At present, these procedures have not been developed to the degree that definitive guidelines for testing and evaluation can be provided for general use. The technology has been developed, however, so that with minor changes in these procedures, substantial improvement can be made in the evaluation process.

Before discussing some recommended changes in the NDT procedures, it is well to review some of the potentials and limitations of NDT. For example, NDT provides data on the behavior of pavements, but to relate behavior to performance, transfer functions are needed that correlate measured individual responses (primarily deflections, in this case) to performance. The performance-evaluation phase of NDT can never be better than the correlation between behaviorial response and performance. Conversely, it is important that the test procedure define the pavement behavior as accurately as possible to decrease the correlation error. It is also important that the pavement behavior be defined in terms that will permit a realistic estimate of pavement response after some specified rehabilitation action, such as an overlay. If it is not possible to predict the behavior of a pavement system after rehabilitation from its properties measured before rehabilitation, then the procedures will be of little value in developing realistic pavement management systems.

With these thoughts as background, the following procedures are recommended for immediate implementation of NDT and evaluation of airport pavements:

1. Conduct NDT of pavements by using both the frequency-sweep and the load-sweep procedures. When using the load-sweep procedure, conduct tests over the load range at both the standard 15-Hz frequency used in the calculation of the DSM and the first resonant frequency.
2. Determine the composition of the existing pavement system through study of the as-built plans and by taking small-diameter [10.2-cm (4-in)] core samples.
3. Determine the resilient moduli of the subgrade by either (a) laboratory tests on undisturbed spoon samples or (b) correlation with soil-classification parameters or degree of saturation of the soils.
4. Calculate a value of effective structural stiffness for the composite structural layers by using the pavement composition and appropriate theory, such as elastic layer or Odemark's.
5. Determine the existing pavement capacity through some or all of the following pavement responses: (a) deflection, (b) critical stresses, or (c) critical strains.
6. Establish the necessary pavement structural system so that the critical responses will be at a level for satisfactory performance under the projected traffic.

This procedure requires expert knowledge of pavement behavior and the factors that influence pavement performance. To use NDT and evaluation at this time without this expert knowledge of pavements is an open invitation to wrong conclusions and expensive failures. To allow such failures at this stage of development may result in loss in credibility for the approach. It is most important to remember that NDT is a tool, not an end in itself, and that tools

are more effective when used by experts.

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