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AN INTRODUCTION TO NONDESTRUCTIVE STRUCTURAL EVALUATION OF PAVEMENTS

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INTRODUCTION

Nondestructive structural pavement evaluation consists of making nondestructive measurements on a pavement's surface and infering from these measurements, insitu characteristics related to the structural adequacy or loading behavior. Such evaluation of highway and airfield pavements is of particular importance to those responsible for the operation and maintenance of these transportation facilities. This information along with knowledge of the skid resistance and riding quality characteristics is essential to the decision maker in the assignment of priorities for his shrinking maintenance dollar.

Significant progress has been made in structural evaluation methodology in the last few decades both in analysis techniques and in equipment. Prior to this time the only procedure available for structurally evaluating an existing pavement was to conduct a series of destructive tests requiring the excavation of test pits combined with field and laboratory tests. Today many agencies structurally evaluate their pavements utilizing nondestructive tests. In some cases these tests require the use of highly sophisticated equipment and analytical techniques supported by high speed computers.

This report contains a description of the current nondestructive testing techniques being used for the structural evaluation of pavements. It contains the following four major sections which can be used to categorize the various testing techniques:

1. Static Deflections
2. Steady State Deflections
3. Impact Load Response
4. Wave Propagation

Each of these sections contains a discussion of the general principles involved in testing as well as specific equipment that is in use. An additional section is presented which describes some of the research and development work which is currently under way to develop improved nondestructive structural evaluation techniques.

STATIC DEFLECTION PROCEDURES

General

Measurement systems that determine the pavement response to slowly applied loads are generally termed static deflection systems to distinguish them from the various dynamic deflection systems now in use and under development. In static measurement systems, loads are applied by slowly driving to or away from a measurement point with a loading vehicle, or, they can be applied by reacting against a stationary loading frame. The most serious problem with this type of measurement technique is the difficulty in obtaining an immovable reference point for making the deflection measurements. Because of this problem, the absolute accuracy of this type of procedure is questionable. The large amount of data developed using these techniques, however, make such procedures an important part of structural pavement evaluation.

The earliest device used for measuring pavement deflection was the General Electric Travel Gage. This instrument was installed by the California Division of Highways as early as 1938 on various highways throughout California and on the Brighton Test Track in 1940 and later, during World War II, on the Stockton Test Track. The installation of these units required the drilling of 5-inch diameter holes through the pavement surface and the insertion of rods to depths of up to 18 feet into the pavement section. Through installations at various depths it was possible to measure not only the total deflection but also the compression contributed by each element of the structural section. It was found that pavement deflection was measurable up to depths of about 21 feet (75). However, the majority of the compression contributing to deflection occurs in the top 3 feet of the structural section.

Because the use of General Electric Travel Gage units was expensive from an installation standpoint and relatively few measurements could be made per day, the need arose for a better deflection measuring device. An improved version, using the linear variable differential transformer (LVDT), was tried at the WASHO Road Test, but difficulties in maintaining calibration persisted. It was not until 1952 when A.C. Benkelman made it possible to measure pavement deflections with the development of his Benkelman beam. Since then, many highway organizations throughout the world have been using this simple device.

In 1954 the California Division of Highways began using the Benkelman beam. While it greatly simplified the task of measuring pavement deflection under wheel loadings, it was, however, still a manually operated, slow, and tedious process. To overcome these difficulties many agencies developed equipment such as the California Traveling Deflectometer and the LaCroix - L.C.P.C. Deflectometer to speed up the data gathering process.

Deflection Response of Pavements

There is a general agreement among pavement engineers that the measurement of the recoverable resilient deflection of a flexible pavement structure can provide valuable information about the structural capacity of the pavement. Based on a review of the literature certain concepts can be expressed concerning the deflection response of a pavement:

1. For adequately designed pavements, the deflection taken during the same season of the year remains approximately constant for the life of the pavement.

2. There is a tolerable level of deflection that is a function of traffic type, volume, and the structural capacity of the pavement as determined by the pavement's structural section. This tolerable level of deflection can be established through the use of the fatigue characteristics of the pavement structure.

3. Overlaying of a pavement will reduce its deflection, and the thickness needed to reduce it to a tolerable level can be established.

4. The deflection history of a well designed pavement undergoes three phases in its behavior (1). A typical curve representing these phases is shown in Fig. 1.

    a. In the initial phase, immediately after construction, the pavement structure consolidates and the deflection shows a slight decrease.

    b. During the functional phase, the pavement carries the anticipated traffic without undue deformation and the deflection remains fairly constant or shows a slight increase.

    c. The failure phase occurs as a result of both traffic and environmental factors. In this phase the deflection increases rapidly and there is a rapid deterioration resulting in failure of the pavement structure.

5. The deflection history of a pavement system varies throughout the year due to the effects of frost, temperature, and moisture. For example, a typical annual deflection history of a pavement subjected to frost action is shown in Fig. 2. It can be divided into four periods (2):

    a. The period of deep frost when the pavement is the strongest.

    b. The period during which the frost is beginning to disappear from the pavement structure. During this period the deflection rises rapidly.

    c. The period during which the water from the melting frost leaves the pavement structure and the deflection begins to drop.

    d. The period during which the deflection levels off with a general downward trend as the pavement structure continues to slowly dry out.

6. For a given flexible pavement structure it is generally known that the magnitude of the deflection increases with an increase in the temperature of the bituminous surfacing material. This is due to a decrease in the stiffness of the bituminous surfacing. The amount of effect varies with the thickness of the bituminous layers and the stiffness of the underlying layers. As the stiffness of the underlying layers increases the effect of increases in temperature on deflection of the total structure decreases. For example, it is thought that almost no temperature correction is required where Portland cement concrete is overlayed with asphaltic concrete, but further research is required to accurately define this. As the thickness of the bituminous layer decreases, temperature changes have less influence on deflections.

7. The use of a static deflection procedure to determine the performance of a rigid pavement has been attempted by a number of agencies with little success. The main problems are the natural tendency of rigid pavements to change their shape throughout a day due to temperature effects (curling) thereby changing the load deflection characteristics and the large deflection basin resulting from loading the stiff rigid pavement structure. This large deflection basin makes it very difficult to obtain a reliable reference point for making measurements.

Most of the deflection procedures in use for the evaluation of flexible pavements rely on the above concepts. These concepts are used by many agencies and provide the users with a rapid, objective, and predictive tool for the evaluation of flexible pavements.

Testing Equipment

Benkelmen Beam

This device shown in Fig. 3 is probably the most widely used method for the measurement of deflection on highway pavements. It operates on a simple lever arm principle. It is used to make either static or quasi-dynamic measurements of either a rebound or loading deflection. When used to make quasi-dynamic measurements the load vehicle (a truck for highway pavements or an aircraft for airfield pavements) is moved past the test point at a creep speed. As the pavement is depressed, the 8-ft-long probe beam pivots around a point of rotation on the reference beam that rests on the pavement surface beyond the area of influence. The portion of the beam behind the point of rotation holds an Ames dial that measures the maximum deflection to within 0.001 in.. When used for static measurements the tip of the probe beam is placed at the point for which the deflection is to be determined, and the load vehicle is moved away from the point, and the maximum rebound deflection is measured. This equipment is very versatile, simple to operate, and inexpensive to obtain. As many as 300 to 400 individual measurements have been made with this device in a working day on highway pavements.

Most highway agencies use an 18,000 lb. axle load on tires inflated to either 70 psi or 80 psi pressure. Only Minnesota uses a lower axle load (14,000 lb.) and then only on pavement systems that are restricted to light loads. The U.S. Air Force uses a loaded aircraft as the loading vehicle, with loads ranging from 15,000 lbs. to 500,000 lbs. depending on aircraft type and the probable structural capacity of the pavement system being evaluated.

Figure 1. Well-designed pavement deflection history curve (1).

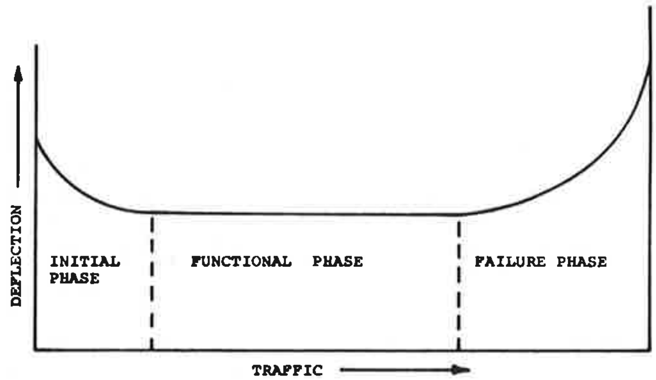


Figure 2. Typical seasonal variations in deflection (2).

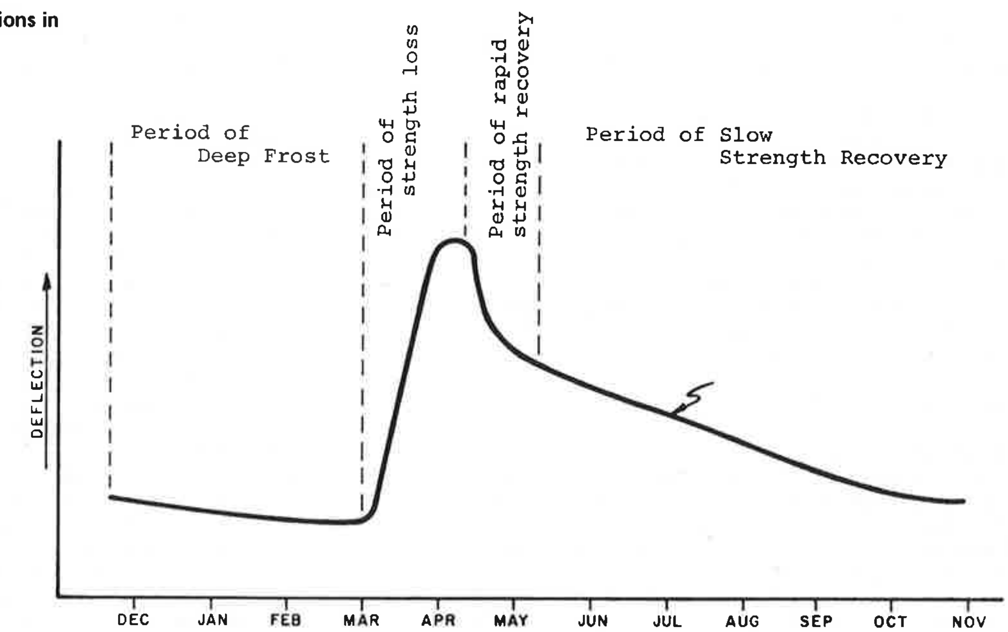


Figure 3. Benkelman beam.

Figure 4. Plate bearing test.

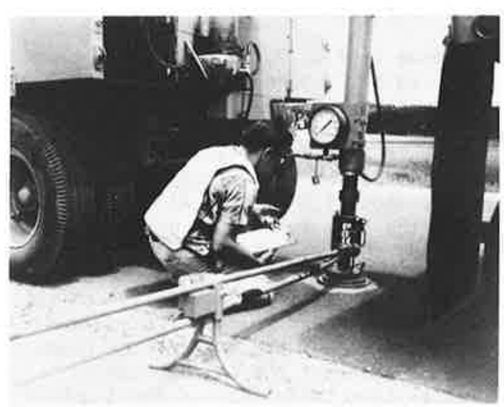


Plate Bearing Test

This procedure, illustrated in Fig. 4, is defined in ASTM D1196-64. It is used primarily as a method for thickness design of a pavement structure. The two organizations now using some form of this procedure are the U.S. Navy and United Kingdom Department of the Environment. It is used by both of these agencies for airfield studies. The procedure is slow, requiring two to three hours for each test point. It has been used by highway agencies but now has been pretty much replaced by other equipment. The U.S. Navy procedure involves the application of load with plates of two sizes (8 in. diam. and 30 in. diam.) to a load that causes a deflection of 0.20 in..

Traveling Deflectometer

This instrument shown in Fig. 5 is used by the California Division of Highways and is an automatic deflection measuring device based on the Benkelmen beam principle. It consists of a tractor-trailer unit that carries an 18,000 lb. single-axle test load on the rear tires and has probes for measuring the pavement deflection between each set of dual wheels simultaneously. The deflectometer is an electro-instrument capable of measuring pavement deflections at 20 ft. intervals as the vehicle moves steadily along the road at a speed of 0.5 mph. The deflections are measured to the nearest 0.001 in. by means of a probe arm resting on the pavement. Between 1500 and 2000 individual measurements can be made during an average working day.

Dehlen Curvature Meter

The Dehlen curvature device consists of a 1/2-in.-thick aluminum bar 1/2-in. wide by 13-in. long with supporting feet at 12-in. centers and a 0.0005-in. dial gage with a 0.05-in. travel, fixed at the center of the bar. Fig. 6 illustrates a longer but similar curvature meter fabricated by the Texas Highway Department. By placing the device between the dual wheels of a loaded test vehicle it is possible to measure the middle ordinate of a curve having a known chord length in the deflected basin, permitting in turn determination of the radius of curvature.

LaCroix - L.C.P.C. Deflectograph

This equipment, shown in Fig. 7, consists of a truck with a rear axle load of 13 tons on twin wheels (3, 4) and with a probe beam similar to the Benkelmen beam. The beam rests on the pavement during the measurement, and it is automatically shifted from one point to the next as the truck travels at constant slow speed of about 1.8 km/hr. At the beginning of the cycle, the beam is at rest on the pavement with the tips of the probe beams ahead of the twin-wheels. As the truck moves forward, the probe arms turn through a very small angle relative to the reference frame. Displacement reaches its maximum when the rear wheels are near the tips of the beams. At the end of the measurement phase, the beam is pulled forward at a speed twice that of the truck until its position relative to the truck is the same as at the beginning of the cycle. When the beam is at rest, the cycle begins again. The vertical displacement of the probe is determined electrically through a displacement transducer. This operation is very similar to that of the traveling deflectometer.

Application

The measurement of a static deflection under a slowly moving load to ascertain the structure capacity of a pavement system must be done with great care and by paying close attention to detail. As pointed out earlier there are a great number of variables involved in the use of this technique. Therefore, it is recommended that any agency planning to use these types of techniques should not attempt to directly adapt another's procedures without thoroughly investigating the basis, both experimental and theoretical.

The following are some procedures in use by various agencies, the listing is not intended to be inclusive of all procedures, but is intended to provide some background on how the static deflection techniques are used.

1. Minnesota: The Minnesota Department of Highways (5,6) uses the Benkelmen beam to establish allowable spring loads and to obtain data for use in the development of overlay designs. They use the Benkelmen beam with a 18,000 axle load distributed between two wheels having 12 ply tires inflated to a pressure of 70 psi. The spring load-carrying capacity is determined by establishing a control section on a relatively weak section of road and using it as an indicator of the loss of strength for the pavements in the area. A deflection versus time curve is developed for the section and the spring load restrictions are imposed as needed to limit deflection.

2. California: The California Division of Highways uses the Benkelmen beam, Traveling Deflectometer, the Dynaflect, and the Dehlen Curvature Meter (7). Their procedures are directed toward the determination of overlay requirements for existing flexible pavement structures. The deflections obtained with the Benkelman beam and the Traveling Deflectometer are both measured under an 18,000 lb. single axle load using dual 12-ply tires inflated 70 psi. The Dehlen curvature meter is used also with the same loading conditions. The radius of curvature is computed by dividing 1.5 by the rebound measurement. The radius of curvature value is converted to a Traveling Deflectometer deflection.

3. Asphalt Institute: The Asphalt Institute method (8,9) uses a Benkelman beam with an 18,000 lb. single-axle load. It requires that 20 random rebound (generally rebound deflections are larger than those obtained by moving the load to the probe) measurements be made each mile which are used to compute the arithmetic mean and the standard deviation for each section of the road. A representative rebound deflection value is computed as follows:

$$d = (\bar{X} + 2s)fc$$

where X = arithmetic mean
s = standard deviation
f = temperature adjustment factor (see Fig. 8)
c = critical period adjustment factor which is to be developed by the user to adjust for the seasonal variations in the pavement strength

4. Ontario Department of Transportation: The Province of Ontario (10,11,12,13) uses the Benkelmen beam with an 18,000 lb. axle load that is imposed on the pavement through dual 12-ply tires inflated to 80 psi. They use their equipment for the development of designs for pavement rehabilitation projects and for evaluation, and follow almost

Figure 5. California traveling deflectometer.

Figure 6. Curvature meter.

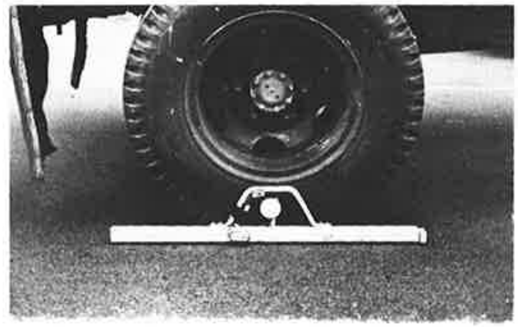


Figure 7. LaCroix deflectograph.

Figure 8. Temperature adjustment factors to 70° F for Benkelman beam deflections as recommended by the Asphalt Institute (8).

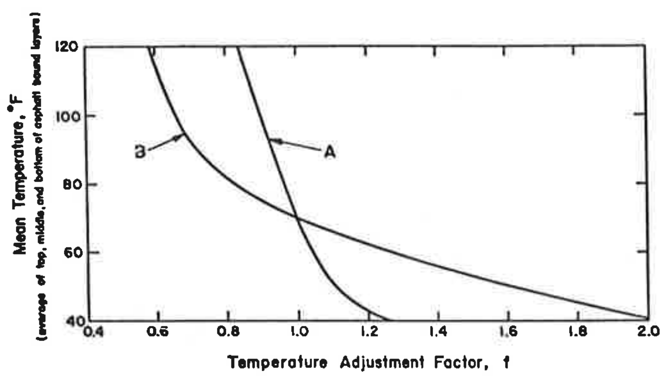
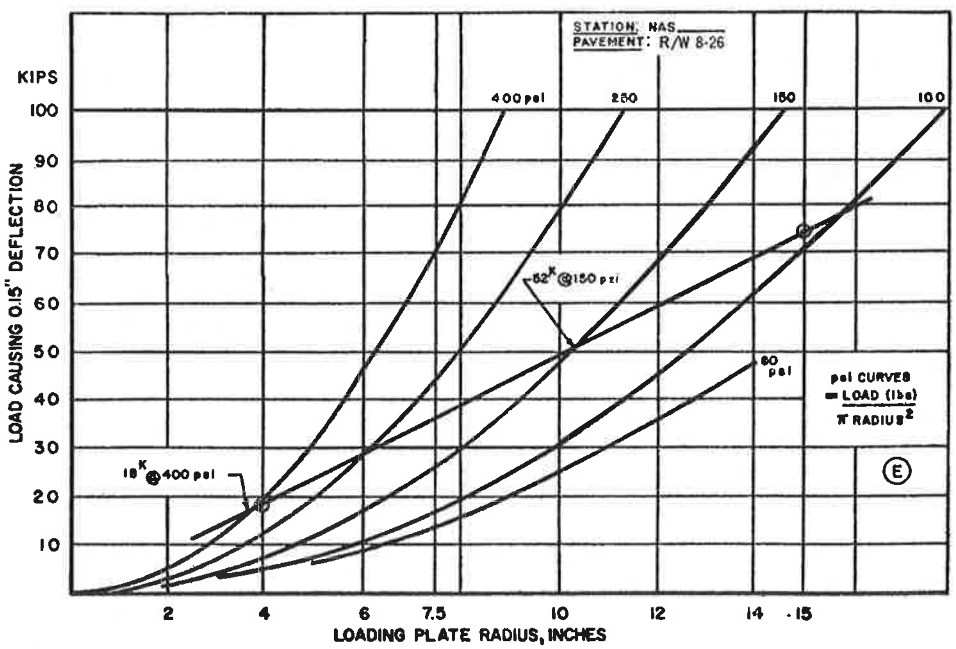


Figure 9. Determination of permissible load rating for flexible pavements using the Navy method (14).



the same procedures as used by the Asphalt Institute but do not correct for temperature; factors, tempered with an engineering judgement, are used to adjust for seasonal variations if measurements are not obtained during the spring thaw.

5. U.S. Navy: The Navy (14) uses a plate load test for evaluation and overlay design. They conduct 8-in. and 3-in. plate load tests on the pavement surface at selected test locations and apply load increments until 0.2-in. deflection is obtained. For overlay design purposes, the top 10-in. of the pavement is removed at a nearby location and a 30-in. diam. plate load test is conducted on the base course. At the same location the base and subbase are removed and a plate load test is run on the subgrade. The data is used with Fig. 9 to compute the load capacity. From the test data, the total load that would cause a 0.15-in. deflection is determined for each of the tests conducted. The loads causing 0.15-in. deflection on the 8-in.-diam. plate are plotted as shown in Fig. 9. These points are connected and the permissible wheel loads at 150 psi and 400 psi tire pressures are indicated.

STEADY STATE DYNAMIC DEFLECTIONS

General

Although there are several different types of steady state dynamic deflection equipment being used to structurally evaluate pavements in the United States, they all have many of the same characteristics. Essentially they all induce a steady state sinusoidal vibration in the pavement with a dynamic force generator. The dynamic force is super-imposed upon the static force exerted by the weight of the force generator (see Fig. 10). The magnitude of the peak-to-peak dynamic force is less than twice the static force to insure continuous contact of the vibrator with the pavement. When one considers the difficulty in obtaining a reference point for deflection measurements, the real advantage of a steady state dynamic deflection measurement system becomes apparent. An inertial reference can be employed to measure dynamic deflections. That is, the magnitude of the deflection change (the peak-to peak value) can be compared directly to the magnitude of the dynamic force change (the peak-to-peak value). For a given value of dynamic force, the lower the deflection, the stiffer the pavement.

Deflections are usually measured with inertial motion sensors. For a pure sinusoidal motion at any specific frequency the electrical output of such sensors is directly proportional to the magnitude of the dynamic deflection. In general, either an accelerometer or a velocity sensor may be used to measure deflections. The latter type is commonly called a geophone and is the type normally employed in dynamic deflection measurements.

The characteristics of a velocity sensor, for example, the Mark Products 2 Hz 1-1H Geophone, are shown in Fig. 11. In this figure, the sensor output per unit velocity is shown for the frequency range of 1 to 100 Hz. The upper curve is for a sensor having a 500 ohm coil resistance and no shunt resistor for damping. Employment of the sensor in this manner results in a high output in the vicinity of the sensor's 2 Hz resonance. The lower two curves are for the same sensor with different values of damping shunt resistors. The horizontal portion on the right of each curve is referred to as the sensor's flat range. In this range, the sensor's velocity response is independent of frequency; thus, the output of the unshunted sensor in volts, divided by 1.05, represents the velocity for any frequency above about 15 Hz.

Fig. 12 shows the output per unit of displacement for the same sensor illustrated in Fig. 11. From this figure it is apparent that the deflection response is highly dependent upon frequency. For example, an output voltage of 2 volts on the unshunted sensor would represent a deflection of about 0.005-in. at 60 Hz or about 0.04-in. at 7 Hz.

It is common practice to employ an electronic integrator to integrate the output of a velocity sensor when it is used to measure displacement. The output:input characteristics of a typical 1 Hz integrator are shown in Fig. 13. With such an integrator, the integration becomes very accurate above about 10 Hz. Thus in the flat range of the sensor's velocity response, the integrated sensor's response becomes flat with respect to displacement. Fig. 14 gives the displacement response of the same sensor shown in Fig. 11 and 12 when its output is integrated with an integrator having characteristics as shown in Fig. 13. The horizontal portion on the right of each curve is the range in which the integrated response of the geophone is proportional to displacement and is independent of frequency; thus, the output of the integrated unshunted sensor in volts, divided by 6.6, represents the magnitude of displacement for any driving frequency above about 20 Hz.

In summary, all steady state dynamic deflection measurement systems employ a dynamic force generator and measure the deflection response of the pavement with inertial motion sensors. For pure sinusoidal motions at any fixed frequency, the output of such sensors are directly proportional to deflection. Thus, to measure deflection it is only necessary to determine the calibration factor (output per unit of deflection) for the measurement frequency. The integrated output of a geophone is the most common type of motion sensor employed when deflections are measured over a range of frequencies. The calibration factor for the output of this type of sensor is constant in its flat response range which generally begins at a frequency value which is about three or four times higher than the resonant frequency of the sensor. If a well integrated output of a geophone is fed into a meter upon which the scale has been calibrated to read deflection, the meter will read correctly only for pure sinusoidal deflections which have a frequency high enough to be within the flat response range of the sensor. For example, if such a meter were used to measure deflections with an integrated unshunted sensor having characteristics shown in Fig. 14 the meter would read correctly above about 20 Hz. It would read about 1.5 times the correct value at 2.5 Hz and about 0.23 times the correct value at 1 Hz.

Steady State Dynamic Response of Pavement Structures

Whenever static loads are applied to the surface of a pavement structure, it deflects an amount which is approximately proportional to the applied force. When the load is removed, it recovers substantially to its former position. Similar pavement behavior occurs in response to dynamic loads, that is, at any specific driving frequency the amplitude of the dynamic deflection is approximately proportional to the amplitude of the applied force. An example of results obtained by Green and Hall (15) using a 16-kip vibrator at three different driving frequencies is shown in Fig. 15. In this series of tests the deflection is almost exactly proportional to load for the tests made at 15 Hz and 40 Hz and somewhat non linear for the tests made at 10 Hz.

Figure 10. Typical output of dynamic force generator.

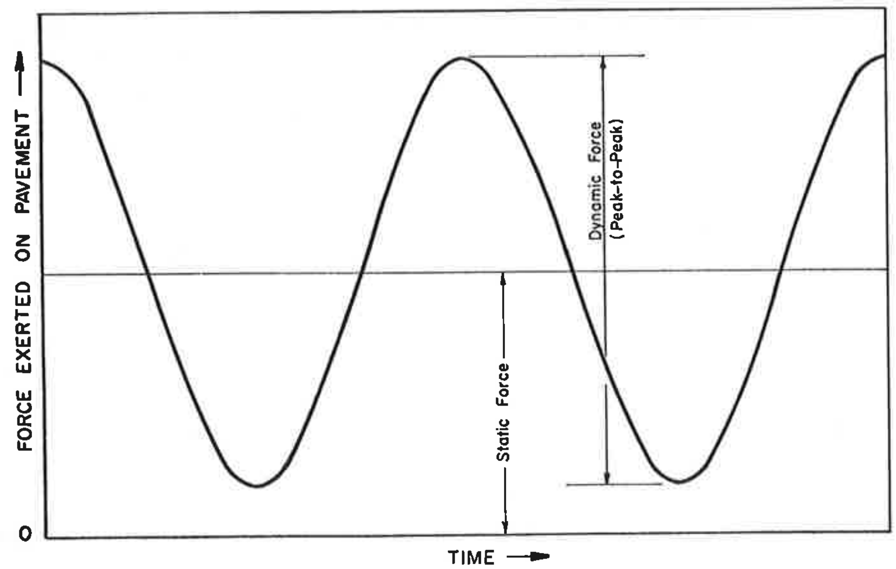


Figure 11. Velocity response characteristics of a 500Ω, 2 Hz geophone.

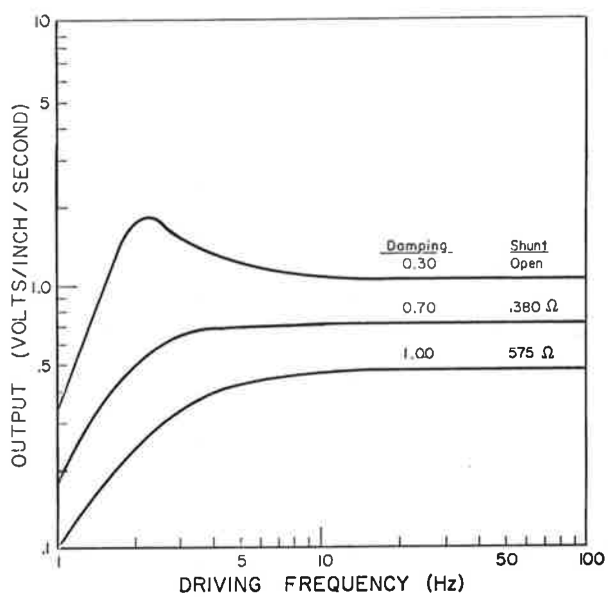


Figure 12. Displacement response characteristics of a 500Ω, 2 Hz geophone.

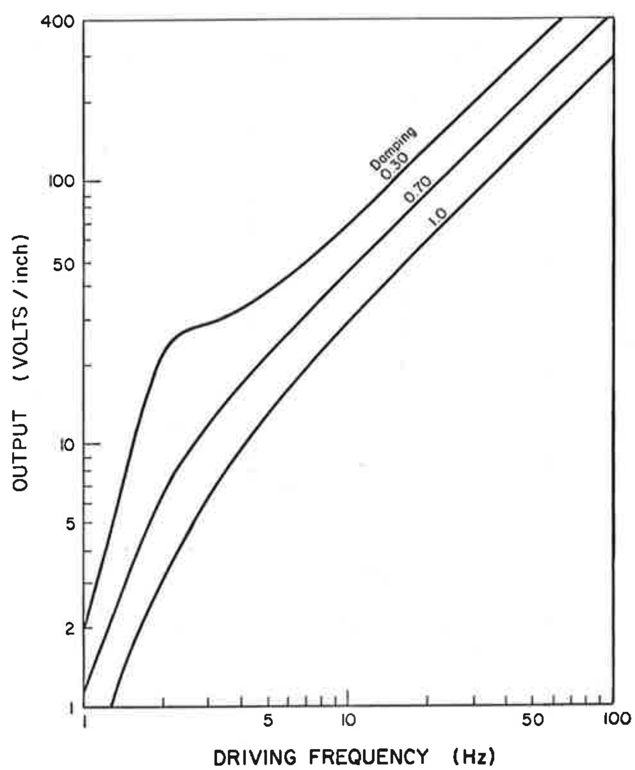


Figure 13. Output: input characteristics of a typical 1 Hz integrator.

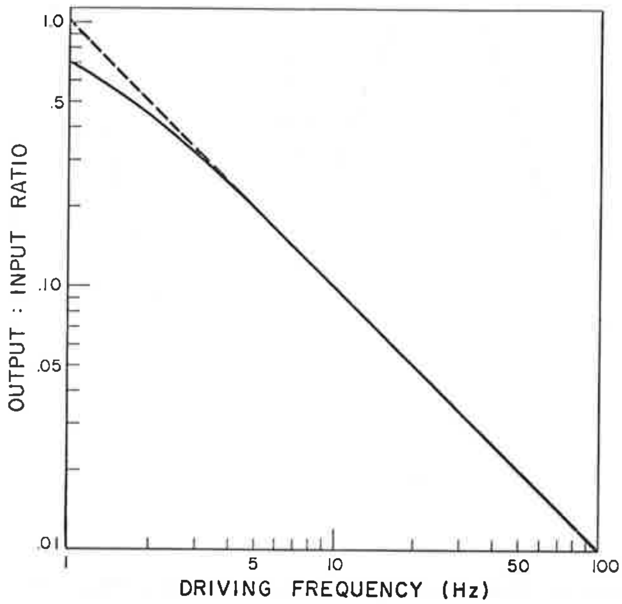


Figure 14. Displacement response characteristics of the integrated output of a 2 Hz geophone.

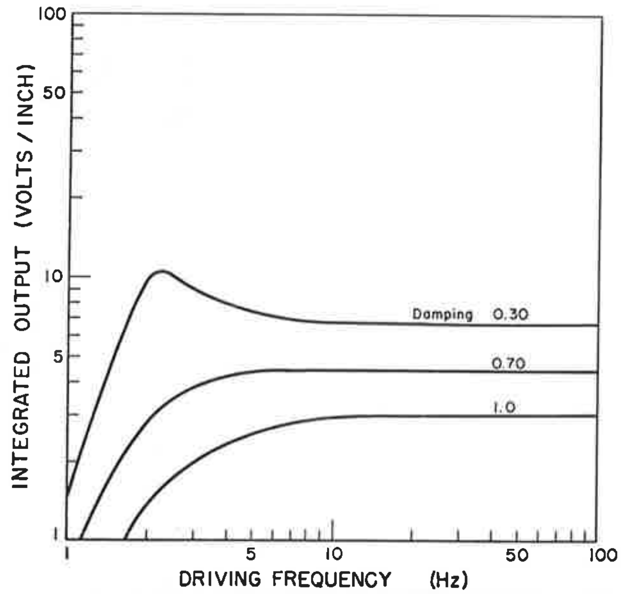
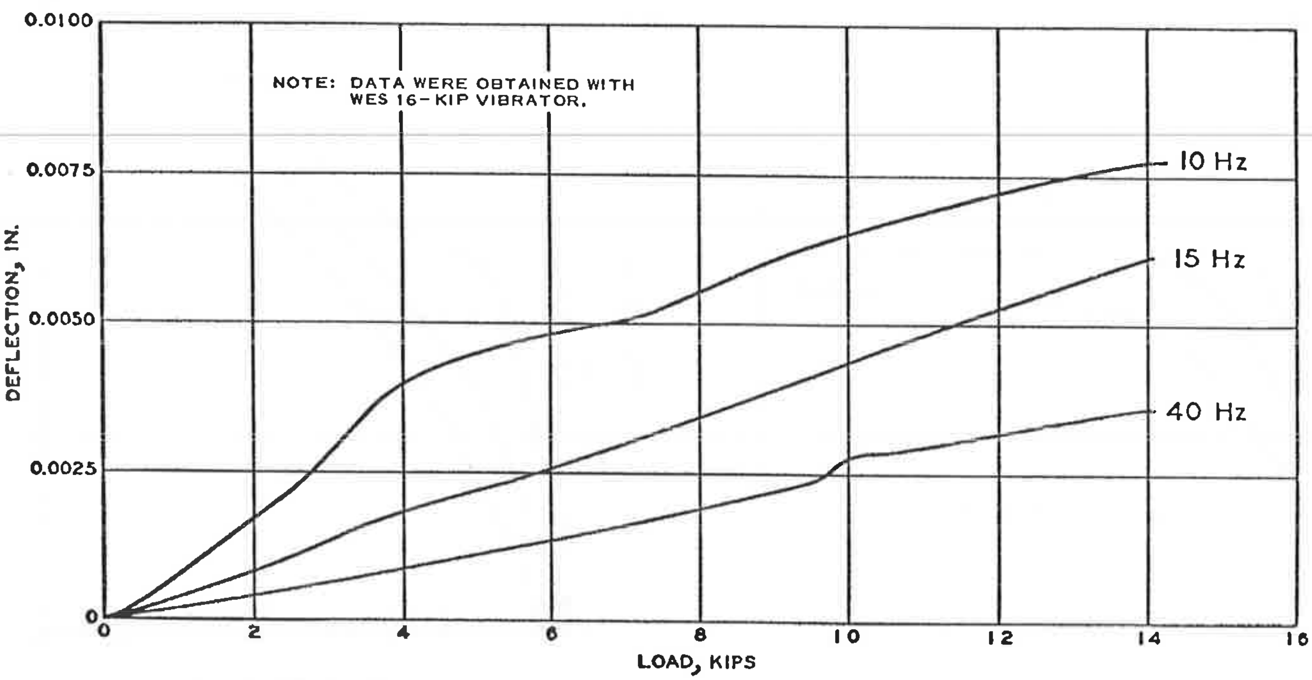


Figure 15. Typical load-deflection curves for frequencies of 10, 15, and 40 Hz.



In 1951 van der Poel (16) introduced the concept of measuring the "overall rigidity of road construction" by dynamic deflections. He defined the overall rigidity, S, as the amplitude of dynamic force required to act on the pavement to produce a unit amplitude in deflection on the surface of the pavement. This term, S, is more commonly referred to now as the "dynamic stiffness." Van der Poel noted that S was not constant but depended upon the driving frequency; S could be expected to increase with increased frequency. He also pointed out the possibility of errors in interpretation due to the existence of unaccounted for resonances and significant differences between the applied force and the actual force acting on the pavement.

In 1953, Nijboer and van der Poel (17) proposed a method for determining the force exerted on the pavement when the force generator was of the eccentric mass type. (This type of force generator produces a force directly proportional to the square of the driving frequency and a correction for the inertia of the force generator must be introduced to arrive at the force exerted on the pavement. Currently it is fairly common practice to monitor the load being induced with a load cell placed as close to the pavement surface as possible to eliminate the need for this correction.)

Assuming the pavement structure to be represented as a mass supported on a viscously damped spring subjected to forced sinusoidal oscillations (also suggested by others [(18,19)]). A complete analytical treatment of this type of mechanical system which is represented in Fig. 16 can be found in most texts on vibrations - e.g., Bolterra and Zachmanoglou (20). The force equilibrium equation for this system is

$$M \frac{d^2x}{dt^2} + C \frac{dx}{dt} + Kx = F_o \ \text{Sin}\ (2\ ft) \quad \ldots\ldots 1$$

where

x = displacement of pavement surface from equilibrium

M = effective pavement mass

C = lumped damping coefficient (force/unit time)

K = static pavement stiffness (force/unit displacement)

F_o= peak amplitude of applied dynamic force

f = driving frequency

t = time

The steady state solution of equation 1 is

$$x = X_o \ \text{Sin}\ (2\pi ft - \phi) \quad \ldots\ldots 2$$

where ϕ, the angular phase lag between the deflection and the applied force, is

$$\phi = \text{arc tan}\ \frac{2\pi fC}{K - 4\pi^2 Mf^2} \quad \ldots\ldots 3$$

and X_o, the peak amplitude of deflection, is

$$X_o = \frac{F_o}{\sqrt{(K - 4\pi^2 Mf^2)^2 + 4\pi^2 f^2 C^2}} \quad \ldots\ldots 4$$

Generally these equations for phase angle and peak amplitude are written in the following form:

$$\phi = \arctan \frac{2\zeta\omega/\omega_n}{1 - (\omega/\omega_n)^2} \quad \ldots\ldots 3a$$

$$X_o = \frac{F_o/K}{\sqrt{[1 - (\omega/\omega_n)^2]^2 + (2\zeta\omega/\omega_n)^2}} \quad \ldots\ldots 4a$$

where

$\omega = 2\pi f$ (angular driving frequency)

$\omega_n = \sqrt{K/M}$ (undamped natural frequency)

$\zeta = C/2\sqrt{MK}$ (damping factor)

The peak amplitude deflection equation is illustrated in Fig. 17. From this figure it is apparent that, regardless of the value of the effective mass (M) or the damping coefficient (c), the dynamic stiffness, F_o/X_o, approaches the static stiffness, K, at low frequencies; i.e., the ratio of dynamic to static deflections for equal force approaches unity at low frequencies and may or may not exceed unity at higher frequencies depending on damping.

Based upon their measurements, Heukelom and Foster (21) concluded that this equation is a good approximation for dynamic deflection testing at low frequencies where the wave length of the surface wave becomes large. Thus in the low frequency range the static pavement stiffness (K), the effective pavement mass (M), and the lumped damping coefficient (C), can be considered constant. In order to represent the pavement with this simple model at higher frequencies they found it necessary to introduce variations into the effective pavement mass. Following this same approach Heukelom (22) reported that for all soils and pavement structures that he had observed the approximation was valid until the driving frequency reached 20 to 35 Hz. Szendrei and Freeme (23) have proposed a more sophisticated seven parameter approximation which consists of two mass-spring-dashpot systems coupled together with either a spring or a dashpot. They found this approximation to fully describe experimental results in the frequency range from 20 to 200 Hz.

Although the dynamic response of a pavement system approaches its static (or elastic) response at low frequencies, exactly what value of driving frequency is low enough to determine the elastic characteristics of a pavement is somewhat in question. As the driving frequency becomes low it becomes difficult to generate dynamic forces and the output of inertial motion sensors becomes very small. These factors combine to make it difficult to obtain accurate, low frequency dynamic deflection measurements.

In 1974, a small experiment addressing the question of how low the driving frequency should be to represent the elastic response of a pavement was conducted at Wyle Laboratories, Inc., El Segundo, California. The force generator was a research model of the Foundation Mechanics, Inc. Road Rater which had the capability of generating a 1250 lb. dynamic force over a frequency range of 4 to 100 Hz. The inertial motion sensor used to measure deflections was a doubly integrated precision accelerometer designed for measuring deflections induced by traffic loads (24). Two different parking lot pavements were tested. One was a non-reinforced concrete pavement about 5 inches thick on a deep sand subgrade. The other was an asphaltic concrete-flexible base pavement section which totaled about 8 inches thick also on a deep sand subgrade. The dynamic force was applied to a rigid steel plate 18 inches in diameter and deflections were measured at a point on the pavement about 5 inches from the plate. The deflection measurements are shown in Fig. 18. On either of these pavements it appears that the static response would be obtained at any driving frequency less than 10 Hz.

The theoretical dynamic response of a three-layered flexible pavement subjected to 1000 lb. peak-to-peak dynamic load on an 8-inch diameter plate was

Figure 16. Mass-spring-dashpot representation of pavement structure subjected to forced dynamic vibrations.

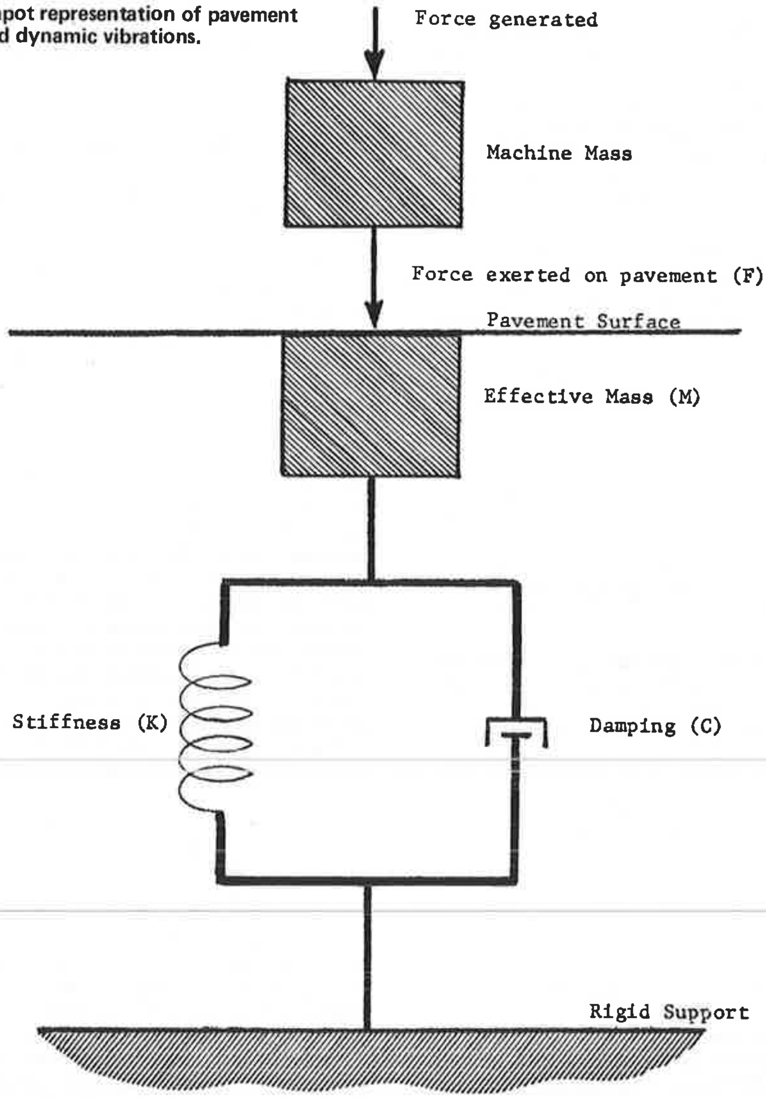


Figure 17. Response of mass-spring-dashpot system subjected to forced dynamic vibrations.

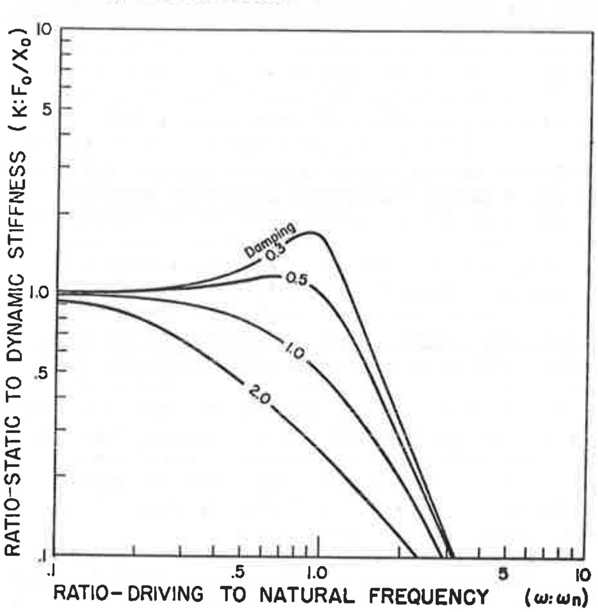
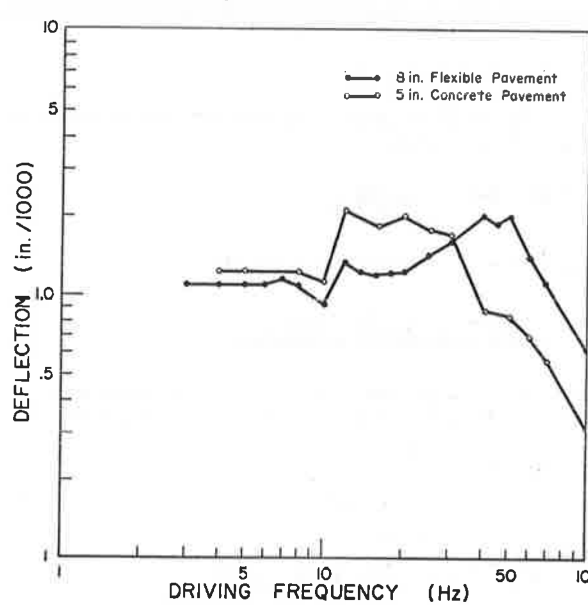


Figure 18. Measured deflection response of pavements subjected to 1250-lb dynamic force.



determined using the CXL450 computer program and a finite element computer program for the analysis of a viscoelastic continuim subjected to oscillatory loading. The thicknesses and material properties assumed in the analysis are given in Table 1.

Fig. 19 shows plots of the surface deflection of the pavement below the load as the frequency of loading is varied from 1 Hz to 100 Hz. The only difference between the two response curves is the value of the subgrade complex modulus. The more elastic subgrade ($E''/E' = 0.034$) produces a very large deflection in the vicinity of 33 Hz. When a more viscoelastic subgrade ($E''/E' = 0.51$), was used, the maximum amplification factor for the pavement deflection dropped to about 1.75. The shape and magnitude of the second curve is strikingly similar to that measured on the flexible pavement in the experiment described above in Fig. 18.

In summary, pavement deflections in response to dynamic loads at any specific driving frequency are approximately proportional to the amplitude of the load. The proportionality factor (or dynamic stiffness) is not independent of driving frequency. Thus when resonances in the force generator system cause forces to be present at frequencies different from the driving frequency, errors in interpretation can result. At low driving frequencies the dynamic pavement stiffness approaches the value of the static (or elastic) pavement stiffness.

Some mention should be made in this section concerning another equivalent method of viewing the steady state dynamic response of a system. It is to determine a system's mechanical impedance, as a function of frequency. Both the input force and the response velocity are determined in amplitude and phase. The ratio of these quantities (force to velocity) in complex representation is defined as the mechanical impedance. This method of representation does not provide any more knowledge about the system than is provided by the displacement and phase information. The method is merely a different way of viewing the same basic response.

Elastic Response of Pavement Structures

Considerable emphasis has been placed upon determining the elastic properties of the layers in pavement structures. Scrivner, et. al. (26) presented an analytic technique for using pavement deflections for determining the elastic moduli of the pavement and subgrade assuming the structure is composed of two elastic layers. Based upon the same assumption, Swift (27) presented a simple graphical technique for determining the same two elastic moduli. Both of these techniques are based upon fitting measured deflections to deflections that would be produced by a point load on a two-layer elastic system. If a load over an area had been used the geometry of the area would enter into the problem and make the solution more difficult. For example, Swift's simple one page graphical solution would require many additional pages to introduce various values of loaded area diameter to layer thickness ratio.

Fig. 20 illustrates the deflection basins that would be produced on a homogeneous elastic medium by a 1000 lb. load distributed over different sizes of circular loading areas. Note that in accordance with St. Venant's principle (28) the deflection basins are quite different. It is clear that the smaller the loading area, the faster the deflection basin approaches the point load case.

Another application of static engineering mechanics theory for the interpretation of deflection measurements has been advanced by Weisman (29). This approach is based upon the Hertz Theory of Plates in the evaluation of pavements. Basically it is the application of the analytical solution of a vertically loaded elastic plate floating on a heavy fluid. The solution to this problem was first presented by Hertz in 1884 and was first applied to concrete pavement analysis by Westerguard in 1926. Since Westerguard's application, this theory has been widely applied to the design and analysis of concrete pavements. Weisman presents a technique for determining the flexural rigidity of the elastic plate - i.e., of the composite pavement - and the density of the fluid subgrade - i.e., the coefficient of subgrade reaction - which will best fit measured deflections.

Neither the two layer elastic theory nor the Hertz theory fit measured deflections on many pavement structures. Nevertheless no readily adaptable analytical procedure exists for the application of more complex engineering mechanics theory for the evaluation of pavement structures. For example, it is apparent that in many cases three layer elastic theory would provide a better fit to measured deflections than that obtained by assuming only two elastic layers. Currently it is possible to accomplish this only by trial and error procedures.

Testing Equipment

As mentioned previously there are several different types of steady state dynamic deflection equipment that are currently being used in the United States for non-destructive structural evaluation of pavements. Two of them are available commercially, the Dynaflect which is manufactured by SIE, Inc., Rt. 5, Box 214, Fort Worth, Texas 76126 and the Road Rater which is manufactured by Foundation Mechanics, Inc., 128 Maryland Street, El Segundo, California 90245. The others have been designed and constructed by agencies involved in pavement research, namely the U.S. Army Waterways Experiment Station, Soils and Pavements Laboratory, Vicksburg, Mississippi 39180; the Eric H. Wang Civil Engineering Research Facility, The University of New Mexico, Albuquerque, New Mexico 87106 and the Koninklijke/Shell Laboratorium, Amsterdam, Netherlands. These agencies will be referred to herein as WES, CERF, and Shell respectively.

Dynaflect

This instrument shown in Fig. 21 is mounted on a small two wheel trailer that can be towed at normal highway speeds by a passenger automobile. To make measurements the vehicle is stopped briefly at a test location where the force generator and the deflection sensors are lowered to the pavement. The operation controls and a meter to read deflections are contained in a control box for convenient access by the driver of the tow vehicle. Thus the driver also serves as the Dynaflect operator (2).

The force generator employs counter rotating masses to apply a peak-to-peak dynamic force of 1000 lbs. at a fixed frequency of 8 Hz. During measurements almost the entire weight of the trailer approximately 1800 lbs. - serves as the static force. About 50 lbs. is transmitted through the trailer hitch to the tow vehicle. The force is applied to the pavement through two, 4-in. wide, 16-in-OD, rubber-coated steel wheels which are spaced 20 in. center to center. The actual contact area of each load wheel is rather small, less than 4 sq. inches.

Deflections are measured with 210Ω, 4.5 Hz geophones that are shunted to a damping factor of approximately 0.7. Normally five sensors are used to make measurements on the symmetry axis which

Table 1. Pavement properties for visioelastic analysis.

Layer	Thickness (inches)	Density (pct)	Complex Elastic Modulus (psi) E' - Real	Complex Elastic Modulus (psi) E" - Imaginary
Surface (Asph. Conc.)	4	108	250,000	66,980
Base	8	100	50,000	1,700
Subgrade	60	100	19,000	646* 9,690†

* The first problem assumed the subgrade to be nearly elastic with an imaginary modulus that was only 0.034 times as large as the real modulus.

† The second problem assumed a more viscoelastic material for the subgrade which had an imaginary modulus that was 0.51 times as large as the real modulus.

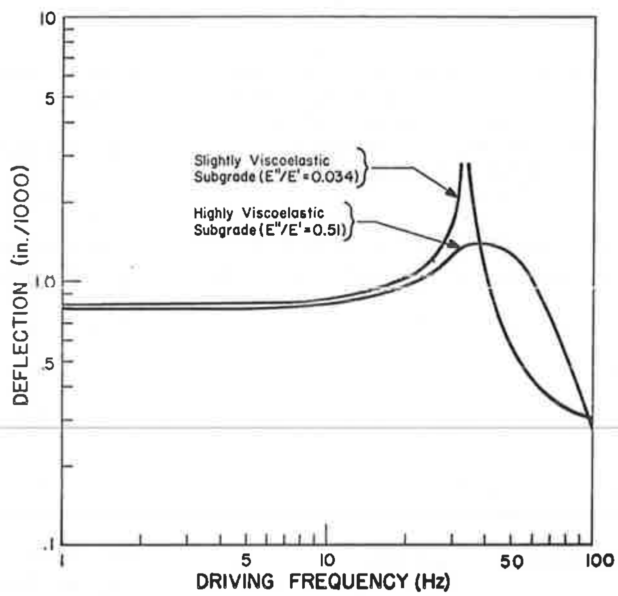


Figure 19. Theoretical deflection response of three-layered viscoelastic pavement structure.

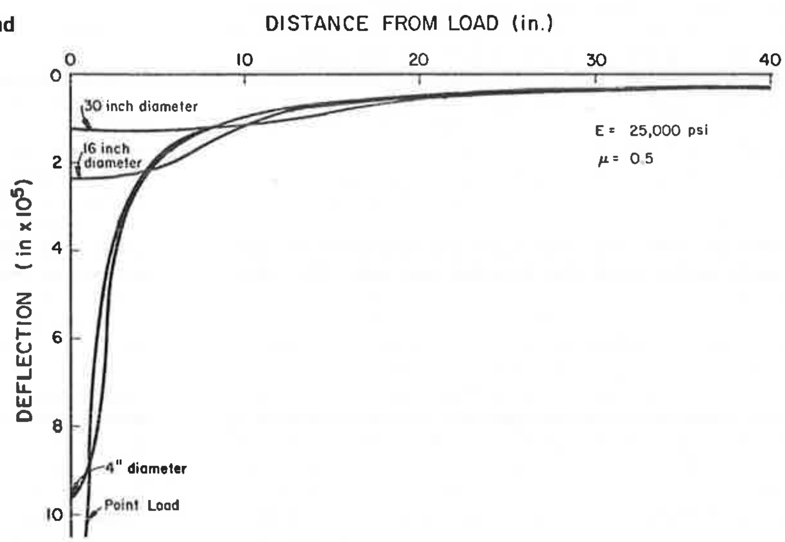


Figure 20. Elastic deflection basins for 1000-lb load uniformity distributed on circular areas.

passes between the load wheels. The deflection for each sensor, in milli inches, is read directly on a meter. Using the Dynaflect calibrator periodic calibration of each sensor though the meter is recommended to compensate for any possible variations in the sensors and associated circulatory.

The Dynaflect is very rapid and simple to operate. The total time required for making a set of five deflection measurements at a test location is about 2 minutes which includes the time required for the lowering and raising of the force generator and deflection sensors.

Road Rater

There are four models of this instrument. Two of the models are designed for mounting on the front of a light duty truck and the other two are mounted in a two wheel trailer (See Fig. 22). To make measurements with either type unit the vehicle is stopped at a measurement location where the force generator and the deflection sensors are lowered to the pavement hydraulically. The operation controls and a meter to read deflections are located inside the vehicle for convenient access by the driver-operator (30).

The force generator for all models consists of a steel mass, hydraulic actuated vibrator. It is capable of producing various magnitudes of dynamic force at driving frequencies between 5 and 100 Hz. All standard models are designed to operate at 5 fixed frequency values in the range of 10 to 40 Hz. At low driving frequencies the maximum peak-to-peak dynamic force that can be produced is limited by the displacement of the hydraulic ram whereas at higher frequencies it is limited by the static force being exerted on the pavement. The dynamic force limits for the force generators used in current models are shown in Fig. 23. Although various other loading plates are available, the loading footprint for all standard models consists of two steel, 4-in. x 7-in. rectangular areas that are spaced 10 1/2-in. center to center. Thus the total contact area is 56 sq. ins.

Deflections are measured using the integrated output of velocity sensors. Normally one or more sensors are employed to make measurements on the symmetry axis which passes between the two loading plates. The deflection of each sensor is read directly on a meter.

The Road Rater is very rapid and simple to operate. The total time required at a test location to make one deflection measurement at a chosen driving frequency is less than one minute. This includes the time required for lowering and raising the force generator and the deflection sensors.

WES 9-kip Vibrator

This instrument shown in Fig. 24 is mounted on a tandem-axle trailer that is towed behind a medium duty truck. Generally a crew of two men is employed, one of which also serves as the driver of the tow truck (31). To make measurements the truck is stopped at a test location where are the force generator is lowered to the pavement through the trailer chassis to the pavement.

The force generator employs counter-rotating masses to apply a dynamic force which is directly proportional to the square of the driving frequency. Although the instrument is designed for operation at driving frequencies from 5 to 60 Hz the normal procedure is to vary it from 5 to 15 Hz. The eccentric counter-rotating masses are preset so that the peak-to-peak dynamic force is about 16,000 lbs. at 15 Hz and it is 1/9 of that value at 5 Hz. When the force generator is lowered its entire weight of 9 kips rests on the pavement. The static and dynamic force is transmitted to the pavement through a set of three load cells that are connected to a 19-in-diameter steel loading plate.

A velocity sensor is mounted directly on the loading plate and the integrated output of the sensor is used for deflection measurements. The actual measurement system consists of an 870Ω, 3 Hz, velocity sensor that is shunted to a damping factor of 0.7 and a 0.8 Hz integrator. The sensor output is recorded along with the output of the load cells on a portable field package that records analog data on light sensitive paper. It is reduced manually.

The WES 9-kip Vibrator is simple to operate. The total time required for obtaining the analog deflection versus load data, where the load is changed by changing frequency, is about 10 minutes which includes the time required for the lowering and raising of the force generator.

WES 16-kip Vibrator

This instrument shown in Fig. 25 is mounted in a 36-ft. semi trailer that contains supporting power supplies and data recording systems. Electric power is supplied by a 25-kw, diesel-driven generator and hydraulic power supplied from a diesel-driven pump which can deliver 38 gpm at 3000 psi. Noramlly a crew of three men is employed one of which also serves as the driver of the semi trailer truck. To make measurements the vehicle is stopped at a measurement location where the force generator is lowered with a hydraulic lift mechanism directly through the floor of the semi trailer to the pavement (31).

The force generator consists of an electro-hydraulic actuator surrounded by a lead-filled steel box. Its total static weight is 16,000 lbs. The actuator uses up to a 2-in. peak-to-peak dynamic force ranging from 0 to 30,000 lbs. at driving frequencies ranging from 5 to 90 Hz. When the force generator is lowered to the pavement its entire weight rests on the pavement. The static and dynamic force is transmitted to the pavement through three load cells that are connected to an 18-in-diameter, steel loading plate. Although the instrument is designed to have a wide range of capability for research investigations, the normal operating procedure is to vary the dynamic force from 0 to 30,000 lbs. at a fixed driving frequency of 15 Hz.

A velocity sensor is mounted directly on the loading plate and the integrated output of the sensor is used for deflection measurements. The actual measurement system consists of an 870Ω, 3 Hz, velocity sensor that is shunted to a damping factor of 0.7 and a 0.8 Hz integrator. The sensor output is recorded along with the output of the load cells on a printer in digital form. In addition the data is plotted on an X-Y recorder to produce a load versus deflection plot. The slope of the upper straight part of this plot is computed and has been termed the DSM (dynamic stiffness modulus) by WES.

The operation of the WES 16-kip vibrator is rapid for obtaining the load versus deflection data of 15 Hz. The total time required is about 3 minutes which includes the time required for the lowering and raising of the force generator.

CERF 6.75-kip Vibrator

This instrument is mounted in a 35-ft. semi trailer that is divided into three separate compartments (32).

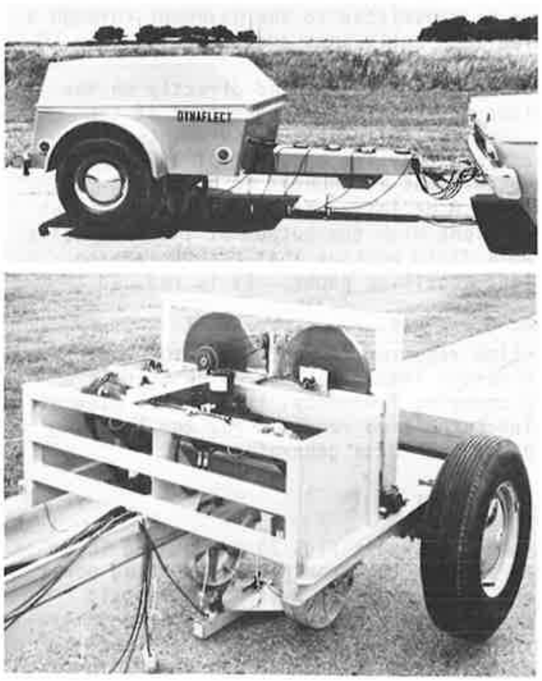


Figure 21. Dynaflect during measurement operations (upper) and with trailer body removed (lower).

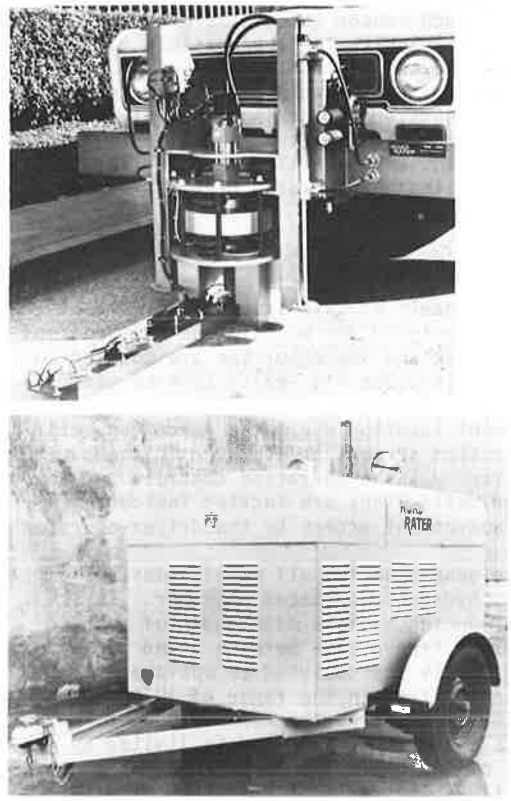


Figure 22. Road rater model 400 (upper) and model 510 (lower).

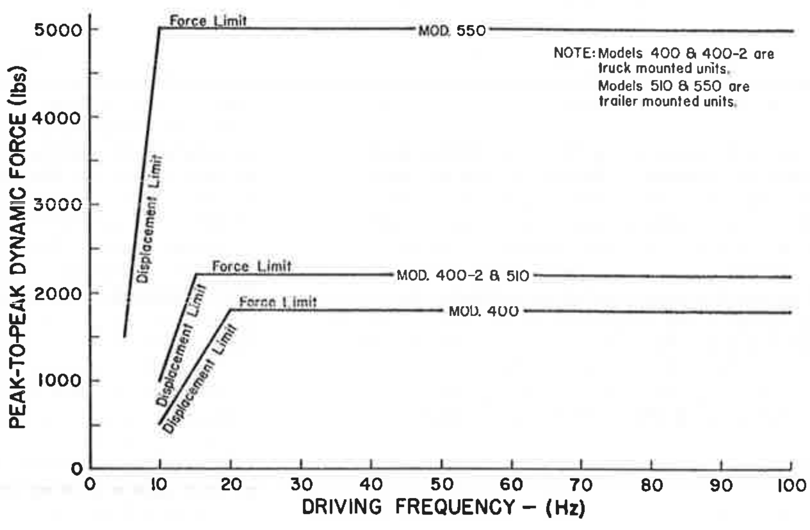


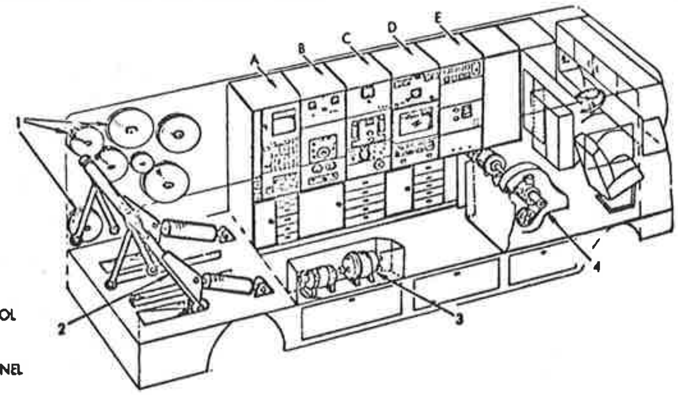
Figure 23. Dynamic force limits of the Road Rater.

Figure 24. WES 9-kip vibrator.

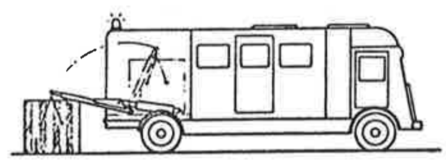
Figure 25. WES 16-kip vibrator.

Figure 26. Shell 4-kip vibrator.

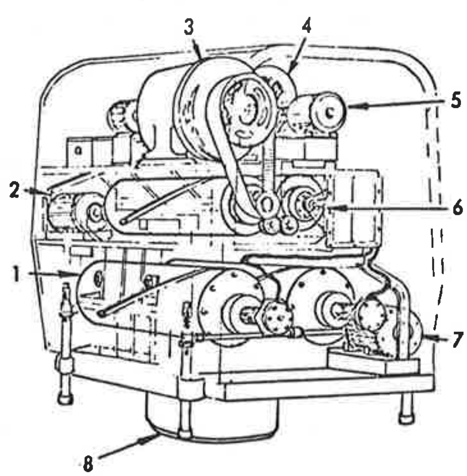
LAYOUT OF INSTRUMENTED TRUCK



FORCE GENERATOR – LOADING/UNLOADING



FORCE GENERATOR



The front compartment contains a 100-kw generator which supplies all required power. The middle compartment contains a transformer, a cooling unit and an electromagnetic force generator. The rear compartment contains the instrumentation and the data acquisition equipment. To make measurements the unit is stopped at a measurement location where the force generator is lowered directly through the floor of the semi trailer to the pavement.

The force generator is an electromagnetic vibrator that weights 6750 lbs.. Its frequency is controlled by a sweep oscillator. A servomechanism on the sweep oscillator is used to hold the load or acceleration at a desired level. The dynamic force and the entire weight of the force generator is transmitted to the pavement through three load cells that are connected to a 2-in-thick, 12-in-diameter, steel load plate.

A velocity sensor and an accelerometer are mounted on the loading plate. The integrated output of the velocity sensor is used to obtain deflection measurements.

The operation of the CERF 6.75-kip vibrator is very similar to the WES 16-kip vibrator. Force versus deflection data for a single frequency can be obtained in about 3 minutes for a single test location. Generally this instrument is used to make measurements over a wide range of frequencies and therefore the time required for measurements is much longer.

Shell 4-kip Vibrator

This instrument, illustrated in Fig. 26 is mounted in a heavy-duty, panel truck that contains all associated electronics, operation controls and recording equipment (33). A three phase ac generator is also contained in the vehicle which supplies all required power. To make measurements the vehicle is stopped at a measurement location where the force generator is placed on the pavement out the rear door of the truck with a hydraulic lift mechanism. The truck is then moved forward to remove any static or dynamic effect that would be transmitted through its wheels.

The force generator actually contains two counter-rotating eccentric mass vibrators. One has a frequency range from 5-20 Hz and the other has a frequency range from 20-80 Hz. The eccentricity of the masses in both vibrators is adjustable so that at any given driving frequency the peak-to-peak dynamic force can be varied from 0 to 8000 lbs. The total static weight of the force generator is slightly more than 4000 lbs. which with the dynamic force is transmitted to the pavement through three load cells that are connected to an 11.8-in-diameter loading plate.

An accelerometer is mounted in a hole through the loading plate. The doubly integrated output of this accelerometer is used for deflection measurements.

Normally the Shell 4-kip vibrator is used to make deflection measurements over a wide range of frequencies and is not considered a rapid measurement system.

Application

All of the steady state dynamic deflection devices can be expected to correlate reasonably well with static deflection measurements. Many evaluation procedures employ these dynamic deflection devices for estimating the anticipated useful life (or load carrying capacity) of pavements based upon the correlation of their measurements with static deflection measurements. Another approach such as that of WES, has been to make a direct correlation between dynamic stiffness and the allowable loads estimated from current thickness design procedures.

As mentioned previously, the major advantage of this measurement technique is that accurate deflection basin measurements can be made with respect to an inertial reference. However, accurate measurements are difficult to make at lower driving frequencies because of the low output of inertial motion sensors as well as the difficulty in generating suitable dynamic forces.

Measurements at low driving frequencies represent the static or elastic response of the pavement. Thus such measurements can be used to estimate elastic constants for major parts of the pavement structure. Such determinations can be used to extrapolate current technology and experience to new and untried materials.

The major disadvantage of these types of measurements is that they represent the stiffness of the entire structure. Although some significant accomplishments have been made in separating the effects of major parts of the pavement structure, the separation of the effects of all of the various layers in the structure with deflection basin measurements is probably impossible. For example, the value of the elastic modulus of a thin surface layer does not significantly effect the characteristics of a deflection basin. Thus, such measurements cannot be used to detect deterioration in such a layer. Even cracking in such a layer does not appreciably influence measurements whereas, it undoubtedly influences the future useful life of the structure.

In addition the parameters that cause plastic deformations in pavement structures are not readily determinable from these types of measurements.

IMPACT LOAD RESPONSE

General

Essentially all impact load testing methods deliver some type of transient force impulse to the pavement surface and measure its transient response. The pavements' transient deflection response is frequently used. In principle, this method of testing is very rapid. The actual duration required for the measurements is at most only a few seconds.

Force impulses are normally generated by dropping a weight from a certain height onto an impact plate which has been placed on the surface of the pavement. The impact plate is designed so that a suitable force impulse is produced. The pavement's response is normally measured with inertial motion sensors like those previously described for use in steady state dynamic deflection testing.

Response of Pavement Structures to Transient Loads

As suggested by Izada (34) a first approximation can be made by representing the pavement structure with the simple mass-spring-dashpot system illustrated in Fig. 16. A complete analytical treatment of this type of mechanical system can be found in many vibrations texts - e.g., Hansen and Chenea (35). The force equilibrium equation for the system is

$$M \frac{d^2x}{dt^2} + C \frac{dx}{dt} + Kx = f(t) \qquad . . . . . 5$$

where

x = displacement of pavement surface from equilibrium,

M = effective pavement mass,
C = lumped damping coefficient (force/unit time).
K = static pavement stiffness (force/unit displacement),
f(t) = force as a function of time, and
t = time.

When the impuse of force is instantaneous with magnitude i, i.e.,

$$F(t) = i\ \delta(t) \qquad \ldots\ldots 6$$

where $\delta(t)$ is the pulse function defined by

$$\int \delta(t)\,dt = \begin{cases} 1 & \text{when integration includes } t=o \\ 0 & \text{when } t=o \text{ is not included.} \end{cases} \qquad \ldots 7$$

the displacement response for the system is given by

$$x(t) = \frac{i}{M\omega_n\sqrt{1-\zeta^2}} \exp(-\zeta\omega_n t)\,\sin(\omega_n\sqrt{1-\zeta^2}\,t). \quad 8$$

where

$\omega_n = \sqrt{K/M}$ undamped natural frequency, and

$\zeta = C/2\sqrt{MK}$ damping factor.

This displacement response for the mass-spring-dashpot approximation of a pavement system subjected to an instantaneous impulse is shown in Fig. 27. When the damping factor, ζ, is less than 1 the system is underdamped, when it is equal to 1, it is critically damped and when it is greater than 1, it is over damped.

As pointed out be Szendrei and Freeme (23) in linear viscoelastic systems there is a direct relationship between impulse testing and steady state sinusoidal testing. Any impulsive force, f(t), which is a function of time, can be represented through the inverse Fourier transform equation as a function of frequency, f, (36), i.e.,

$$f(t) = \int_{-\infty}^{\infty} F(f)\exp(j2\pi f t)\,df \quad \ldots\ldots\ldots 9$$

where $j=\sqrt{-1}$ and F(f) is the Fourier transform of f(t) defined by the Fourier equation, i.e.,

$$F(f) = \int_{-\infty}^{\infty} f(t)\exp(-j2\pi f t)\,dt \quad \ldots\ldots\ldots 10$$

$F(f)$ is the peak amplitude of the steady state sinusoidal force input at any frequency, f, where the sum (or integral) of the sinusoidal inputs over all frequencies is precisely equivalent to the impulsive force input, F(t).

Similarly the transient deflection response, X(t), due to the inpulsive force, F(t), is related to $X(f)$, through the inverse Fourier transform and the Fourier integral equations. Thus $X(f)$, is the peak amplitude of the steady state sinusoidal deflection response to the input force, $F(f)$, where the sum of the deflection responses over all frequencies is equivalent to the transient deflection response, X(t).

If x(t) and f(t) are determined by measurements, their Fourier transforms can be determined analytically or by numerical integration. The ratio of these Fourier transforms, $X(f):F(f)$, is the peak displacement response due to a steady state sinusoidal force of unit peak amplitude as a function of frequency, f.

For example, the Fourier transform of the instantaneous impulse of force defined by equation 6, is (36)

$$F(f) = i \qquad \ldots\ldots\ldots 11$$

and the Fourier transform of the resultant displacement response defined by equation 8 can be evaluated analytically to be

$$X(f) = \frac{i/K}{1-(f/f_n)^2 + j2\zeta f/f_n} \qquad \ldots\ldots\ldots 12$$

where

$$f_n = \sqrt{K/4\pi^2 M}$$

Equation 12 can be written in the form of magnitude and phase angle as follows

$$X(f) = \frac{i/K}{\sqrt{[1-(f/f_n)^2]^2 + (2\zeta f/f_n)^2}} \qquad \ldots\ldots 12a$$

$$\theta = \arctan \frac{2\zeta f/f_n}{1-(f/f_n)^2}$$

Thus the peak displacement response due to a steady state sinusoidal force of unit peak amplitude as a function of frequency is

$$\frac{X(f)}{F(f)} = \frac{1/K}{\sqrt{[1-(\omega/\omega_n)^2]^2 + (2\zeta\omega/\omega_n)^2}} \qquad \ldots\ldots 13$$

$$\phi = \arctan \frac{2\zeta\omega/\omega_n}{1-(\omega/\omega_n)^2}$$

where

$\omega = 2\pi f$

ϕ = phase angle between the applied force and its deflection response

This is the same result shown in the previous section as the steady state solution of an applied sinusoidal force of angular frequency, ω, and a peak amplitude, F_o. See equations 3a and 4a. This example serves to illustrate the relationship between impulse and steady state testing.

Because the Fourier transform of an instantaneous impulse contains equal amounts of all sine waves from f=0 to f=∞, the Fourier transform of its deflection response provides complete information concerning the steady-state frequency response. However, in practice it is impossible to generate instantaneous impulses. Nevertheless, if impulses are generated that are short compared to the rise time of the pavement, the magnitude of the impulse of force, $\int f(t)dt$, and not its shape is all that is important. The Fourier transform of the displacement response to such short pulses will contain the steady state frequency response of all frequencies that are of practical significance. The rise time as used here is the time required for the pavement to deflect from 10 to 90 percent of its maximum deflection after being subjected to a step loading. Based upon the observations of Szendrei and Freeme (23) it appears that the rise time can be expected to be in the order of 3 to 6 msec. Thus, the force impulse should be 1 msec or shorter, to consider it as an instantaneous impulse. Longer force impulses will not contain all steady state frequencies which produce significant responses. The deflection response of any type of longer force impulse will contain information only about those frequencies that are contained in the Fourier transform of the force impulse. Thus shape, magnitude and duration of the force impulse significantly affect the response of the pavement.

The duration of an impulse of force generated by dropping a weight on a surface depends upon many factors, including the mass and geometry of the dropped weight. Under some conditions the duration of such force impulses may be several milliseconds and, therefore, not short compared to the rise time of the system response. Such a force impulse will have a frequency distribution in its Fourier transform that is maximum at zero and falls off as frequency increases. The peak of the force impulse divided by the peak of the deflection response can be taken as a measure of the pavement stiffness which is more-or-less an average for the low frequency range. The shorter the impulse, the wider is the frequency range which is represented. If all force impulses are alike in shape, magnitude and in duration this approach provides a means of measuring an overall pavement stiffness.

Testing Equipment

Impact testing has not been used extensively in the United States for pavement evaluation. The instruments that have been used were designed and constructed by agencies involved in pavement research, namely the Cornell Aeronautical Laboratory, Transportation Research Department, Buffalo, New York and the Washington State University, College of Engineering Research Division, Pullman, Washington 99163. These agencies will be referred to as CAL and WSU respectively. An impact instrument called the Phoenix Falling Weight Deflectometer is available commercially from A/S Phoenix Tagpap og Vejmaterialer, DK 6600 Vejen, Denmark.

CAL Impulse Testing

In the early 1960's the Cornell Aeronautical Laboratory, CAL, investigated the feasibility of impulse testing for detecting seasonal variations in the load-carrying capacity of flexible pavements (34). The instrumentation consisted of an impulse generator and several different sensors to measure the pavements response.

The trailer mounted force generator is shown in Fig. 28. It is equipped to raise and release a 500 lb. steel bar onto a 1-in-thick, 15-in-diameter, aluminum striker plate. Although the drop height is adjustable from 0 to 4 ft., a drop of 1 ft. was used in their experiments. The duration of the force impulse was found to be somewhat variable, between 3 to 6 msec, depending upon the type of pavement being tested. The most meaningful measurements of the pavement response was made with an inertial motion sensor named, Dynamic Displacement Transducer, DDT. The DDT was designed and constructed by CAL. It is basically a seismic mass displacement sensor that has a natural frequency of about 1.7 Hz.

It was concluded in this preliminary investigation that the impulsive loading technique offered a possible means of estimating load carrying capacity. The first peak deflections determined with the DDT, appeared to be the most promising of the response variables measured.

WSU Impulse Testing

Recently the Washington State University, WSU, has also been investigating the feasibility of impulse testing for non-destructive pavement evaluation (37,38,39). Their tests indicated that the structural parameters of pavements are linear, or sufficiently linear over a broad enough range that the forces utilized for pavement structural evaluation need not necessarily be large. Thus evaluation equipment need not be large and heavy. Based upon the fact that a force impulse contains a spectrum of frequencies, it is concluded that impact testing offers an advantage over single frequency excitation. Single frequency excitation risks the hazard of the response being adversely affected when driving frequency is in the neighborhood of a sharp resonance.

The instrumentation consists basically of a hammer capable of delivering a blow of controlled energy to the pavement and two transducers to measure the pavements response. Two piezoelectric accelerometers are used which were designed and constructed by WSU. These transducers were designed to obtain a high output voltage with good low frequency response. One accelerometer is placed on the pavement very close to the point of impact. The second is positioned 18 inches further away. The electrical output of each accelerometer is rectified and integrated to yield the time integral of the absolute value of the accelerometer output. The resulting processed output from the first transducer is designated R_1 and from the second transducer R_2. An Impulse Index has been designated as R_1^2/R_2. From results to date the results appear to have a good correlation with Benkelman Beam and Dynaflect measurements. The final report of the current testing program is scheduled for release in the near future.

Actually two different applications of the instrumentation are being investigated by Washington State University. Both types are shown in Fig. 29. One is a suitcase version whereas the other is a vehicle mounted version which can travel down the highway at reasonable speed and make measurements every few feet.

Phoenix Falling Weight Deflectometer

This instrument, shown in Fig. 30, is mounted on a small two wheel trailer that can be towed behind a passenger automobile at normal highway speeds (40). To make measurements the two vehicle is stopped at a test location where the driver-operator raises the unit from its horizontal transport position into its vertical operating position and then lowers it to the pavement by opening a hydraulic valve. The deflection reference beam is placed on the pavement with its LVDT feeler in direct contact with the pavement through a small hole in the units loading plate. The falling weight is raised to the proper drop height by means of a hand operated hydraulic pump where it is released automatically.

The falling weight, which weighs 150 kg (330 lbs) is normally dropped 40 cm (15.7 in). It falls onto a spring damping system that produces a force curve closely approximating a half-sine wave. The duration of the force impulse is about 26 msec and its peak magnitude is 5 metric tons (5.5 tons). This

Figure 27. Response of mass-spring-dashpot system to an instantaneous impulse of force, i.

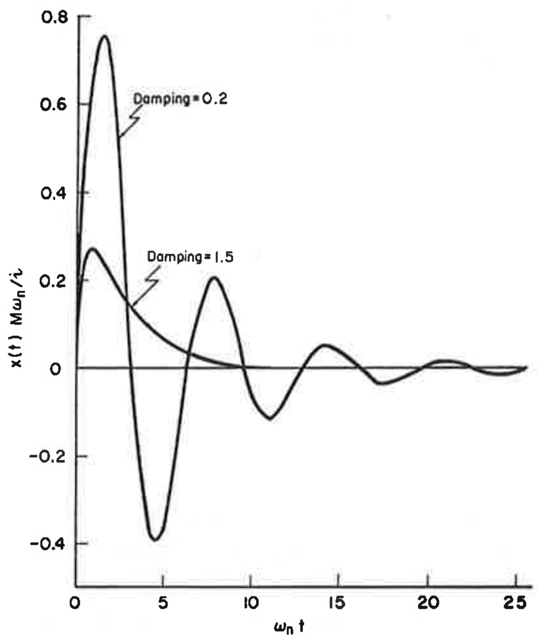


Figure 29. WSU impulse devices—suitcase model (upper) and vehicle model (lower).

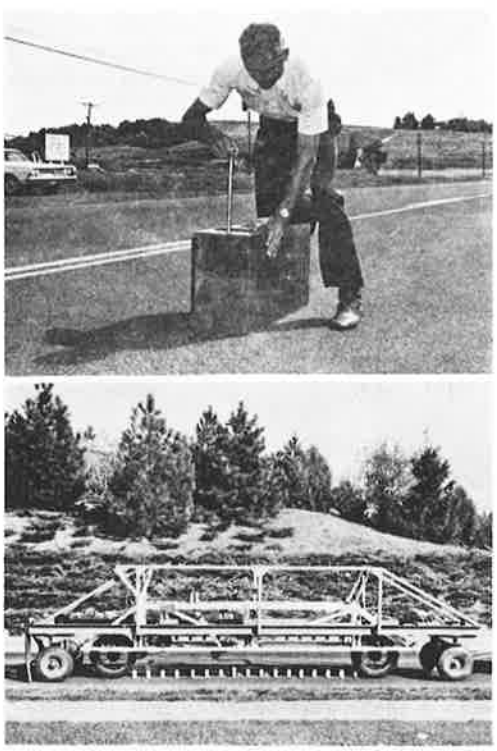


Figure 28. CAL impulse generator.

Figure 30. Phoenix falling weight deflectometer.

Figure 31. Schematic representation of Rayleigh wave motion.

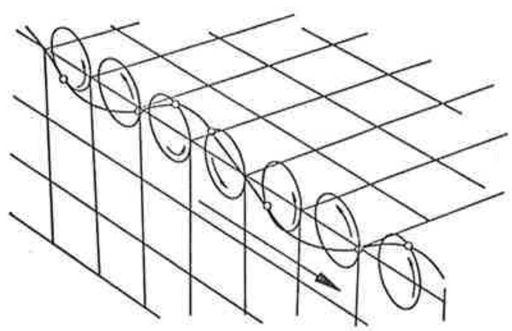
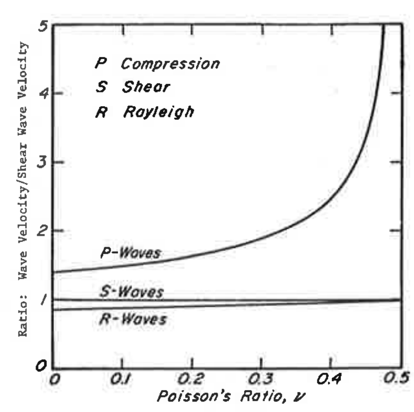


Figure 32. Effect of Poisson's ratio on wave velocity.



peak value is directly proportional to the square root of the drop height and smaller values can be used. The loading plate normally employed is rubber coated to distribute the load impulse evenly over its 30 cm (11.8 in) diameter surface.

An LVDT, supported by a cantilever beam system is used to measure deflections. Its output is recorded on a time base recorder or the peak deflection value is read directly on a peak voltmeter.

The device is simple to operate. The total time for making a set of three replicate measurements at a test point is about 5 minutes. This time includes raising the unit from its transport position, making the measurements and lowering the unit again.

Application

All of the impact load devices can be expected to provide information that will correlate reasonably well with static deflection measurements. However, research has not yet progressed far enough for evaluation procedures to be developed sufficiently for large scale implementation in the United States.

By far the major advantage to this testing approach is that the actual duration required for the measurements is almost only a few seconds. In addition, the response data available during this short period contains the same information that is contained in a number of steady state deflection tests.

Disadvantages include the problem of obtaining accurate response information in the low frequency range because of the characteristic low output of inertial motion sensors. Also, to obtain reliable response information in the significant frequency range of the pavement requires large force impulses which have a short (less than 1 msec) duration. Such impulses are difficult to produce. Nevertheless considerable pavement characterization information can be obtained when force impulses of longer duration are used. To obtain this information requires Fourier analysis of the input and output as described previously. Not much research has been reported based upon this approach.

Essentially the impact testing technique is like any type of deflection testing in that it represents characterization of the entire structure. The techique does not provide information that readily separates the effects of its various layers. In addition the parameters that cause plastic deformation in the structure are not readily determinable from impact testing.

WAVE PROPAGATION TECHNIQUES

General

Wave propagation techniques offer methods for the determination of the elastic properties of individual pavement layers and subgrades. This information is of particular interest for use with theoretical layer analyses of pavement systems. Measurement of the velocities of waves propagated through pavement and soil layers began in the late 1930's (41,42) with the work of the German Society of Soil Mechanics. Similar work has been continued by the Transportation and Road Research in South Africa, the Civil Engineering Research Facility in New Mexico, the Waterways Experiment Station, and others. In spite of the extensive efforts of these organizations, there still remain many aspects of the vibratory wave propagation techniques which are not clearly understood.

There are two basic techniques for propagating waves through pavement structures--(1) steady-state vibration tests and (2) seismic (impulse) tests. The wave propagation method involves the measurement of the velocity and wave length of the surface waves propagating away from the vibratory source placed on the surface. Generally, three types of waves are transmitted when a pavement surface is subjected to vibration source:

1. compression (P),
2. shear (S), and
3. Rayleigh (R).

The P and S waves are body waves where the R wave is a surface wave. The dissipation of energy input from a vibrator on a semi-infinite half-space has been found to be distributed as P, S, and R waves in the following amounts (43):

Wave Type	Percent of Dissipated Energy
P	7
S	26
R	67

From the above data it can be concluded that the Rayleigh waves are the dominant waves being transmitted. Also, P and S waves attenuate rapidly with radial distance from the vibration source. For these reasons, Rayleigh waves are the type of waves measured in the wave propagation technique. A schematic representation (44) of the Rayleigh wave is presented in Fig. 31.

The shear wave travels at the velocity given by:

$$C_S = \sqrt{\frac{G}{\rho}}$$

where:

G = shear modulus
ρ = mass density

while the compression wave velocity is:

$$C_P = \sqrt{\frac{\lambda + 2G}{\rho}}$$

where:

$$\lambda = \frac{E\,\nu}{(1 + \nu)(1 - 2\nu)}$$

where:

E = modulus of elasticity
ν = Poisson's ratio

and the Rayleigh wave velocity is given by:

$$C_R = \alpha\, C_S$$

where α is a function of Poisson's ratio and varies from 0.875 for $\nu = 0$ to 0.955 for $\nu = 0.5$. A relationship between a shear wave velocity and a compression wave and Rayleigh wave velocities as a function of Poisson's ratio is shown in Fig. 32. Fig. 32 shows the Rayleigh and shear waves to be almost identical and not particularly dependent on Poisson's ratio. However, the value of the compression wave velocity is strongly dependent on Poisson's ratio.

Measurement Techniques

Field test procedures for the wave propagation measurements involve two general types of tests. Rayleigh wave velocities are determined from steady-state vibratory pavement response and compression wave velocitities are measured from impulse (seismic) tests. A laboratory test is sometimes used for correction of the strain level in the subgrade material.

Laboratory procedures are available for the determination of the elastic properties of pavement and soil specimens using wave propagation techniques. The laboratory procedures, however, require that samples of the pavement material be obtained for testing, and therefore, may not be considered as a nondestructive technique. The laboratory procedures that parallel the field vibratory procedures are considered worthwhile of reporting here. The methods may be applicable to pavement design. Two procedures, the resonant column method and the pulse method, will be described.

Steady-State Vibration Tests

From an experimental standpoint, the wave propagation method requires that the velocity and wavelength measurements be obtained over a wide frequency, 5 to 50,000 Hz, if elastic properties of all the pavement layers are to be determined. Various types of vibrators have been employed. Early work by the Royal Dutch Shell Laboratory, the Road Research Laboratory, and the Waterways Experiment Station (WES) utilized a mechanical vibrator for low-frequency vibrations (5-100 Hz) and a small electromagnetic vibrator for the high-frequency work.

The general procedure is to place the vibrator on the pavement surface and set the equipment in operation at a constant frequency. A schematic of the test setup is shown in Fig. 33. A measuring tape is set up in the direction that the measurements are to be taken. A transducer (either accelerometer or velocity-type) is then placed on the pavement surface as near to the vibrator as possible. By means of an appropriate phase-marking circuit and an oscilloscope, the sine wave is monitored, and the phase mark is adjusted until it is at a peak or trough point on the sine wave as shown in Fig. 34. Once this is done, the transducer is moved away gradually, and the phase mark shifts away from its position until it coincides with the succeeding peak or trough. (Any other point on the wave form could be used.) When this occurs, the distance is noted. The distance between the phase mark coinciding with successive peaks or troughs is one wavelength. In order to obtain an average value of the wavelength, the transducer is moved a sufficient distance to record several wavelengths. The number of wavelengths is plotted against the distance as shown in Fig. 35 in order to obtain the average wavelength for each test frequency. The velocity of the Rayleigh wave (The propagated wave in a layered system is not exactly a Rayleigh wave (45) by definition since a Rayleigh wave is defined in a homogeneous elastic half-space. However, Rayleigh wave is used in this report to describe the measured surface wave using wave propagation techniques.) is determined from:

$$C_R = Lf$$

where:

C_R = Rayleigh wave velocity
L = wavelength
f = frequency

The frequency is then changed and the procedure repeated until adequate information is obtained. Since the wavelength is a function of frequency, lower frequencies (less than 200 Hz) are generally required to produce longer wavelengths to obtain data from the lower velocity materials which occur in the lower pavement structure. It has been the practice of many investigators to use combinations of light high-frequency vibrators and heavier low-frequency vibrators. Table 2 summarizes the equipment being used by some investigators.

The above procedure was somewhat improved (46) through the use of a circular time base on the oscilloscope. The display on the oscilloscope consists of a simple bright spot whose position on the circular time base indicates the relative phase difference between the vibrations at the oscillator and at the pickup. The circular time base makes one complete revolution per cycle of the applied vibration so that the spot on the oscilloscope display moves once around a graduated circle as the transducer is moved a distance of one wavelength. This technique eliminates the need of searching for the particular distance that gives an exact wavelength.

Another test procedure, which involves mounting two or more transducers on the pavement surface at known distances apart, has been used by the Civil Engineering Research Facility (CERF). A schematic of the equipment setup for this procedure is shown in Fig. 36. Only one vibrator is used for the full desired frequency range. The vibrator is then swept through the desired frequency range and the phase difference between any two transducers is continuously recorded as shown in Fig. 37. The data are recorded on magnetic tape for automatic reduction through the use of computer facilities. The wavelength for any desired frequency can be calculated from:

$$L = \frac{360d}{\theta}$$

where:

L = wavelength
d = spacing between transducers
θ = measured phase difference between transducers for distance d

The wave velocity is obtained from

$$C_R = f \frac{360d}{\theta}$$

It has been noted that the dynamic modulus obtained by wave propagation tests is the tangent modulus value at very low strain levels. Therefore, in order to make comparisons between measured and theoretically computed deflections using the modulus value as determined from dispersion curves, a correction technique for nonlinearity of the stress/strain relationship was developed (47). Soil specimens are tested in the laboratory under vibratory loadings and relationships between shear stress and shear strain are obtained. A reference strain value is computed based on soil type, moisture content, and density. From the relationships obtained between shear modulus and shear strain, the maximum shear modulus at any point in a stressed pavement system can be corrected depending on the strain level at that point.

Impulse Tests (Seismic Tests)

Seismic tests may be conducted in order to determine the velocity of compression waves, which can be used with the shear wave (or Rayleigh wave) velocity to compute Poisson's ratio. However, this

Figure 33. Schematic diagram of equipment for wave propagation test.

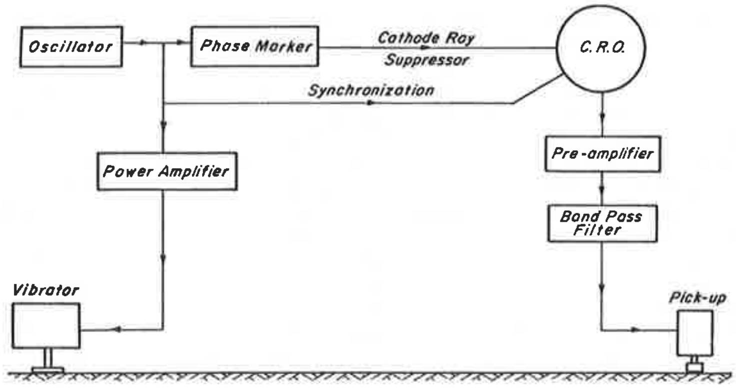


Table 2. Wave velocity vibration test equipment.

Equipment User or Developer	Type Device	Frequency Range	Static Weight lb	Dynamic Force p-p lb	Contact Plate Diameter Size, in.	Time Required per Test
Transportation and Road Research Laboratory	(1) Electromagnetic (2) Magnetostrictive	25 Hz - 30 kHz	30 1	10 Very small	4 1	15 min - 2 hr
Air Force Weapons Laboratory	Electromagnetic	5 - 3500 Hz	6,750	5,500	12	15 min
Commonwealth Scientific and Industrial Research Organization, Victoria, Australia	Torsional Radiator	30 Hz - 9000 Hz	46	3.5 newton meter (torque)		
Laboratoire Central des Ponps et Chaussèes, Paris, France	Electromagnetic	40 Hz - 10,000 Hz	31	2.5		
Waterways Experiment Station	(1) Electrohydraulic (2) Electromagnetic	5 - 100 Hz 50 - 5000 Hz	16,000 300	30,000 500	18 Variable 4 - 12	15 min - 1 hr
National Institute for Road Research, South Africa	(1) Eccentric Mass (2) Electromagnetic	10 - 75 Hz 15 - 5000 Hz		2×10^4N 200N		

Figure 34. Sine wave signal with phase mark.

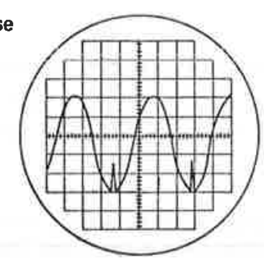
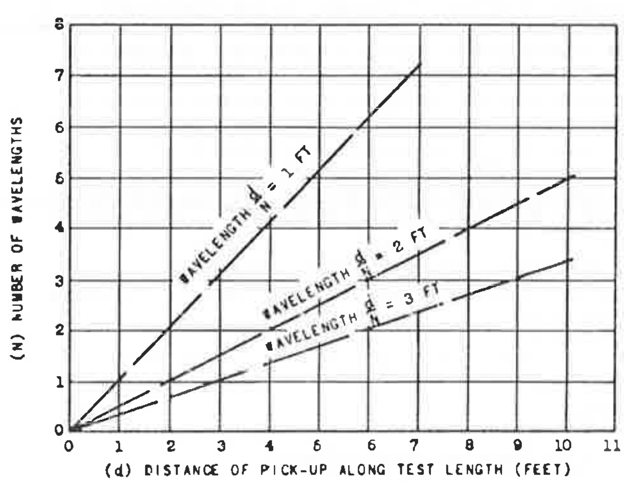


Figure 35. Wave length-distance relationship.



is only good for soils where the velocity of the materials increases with depth, and is not applicable to layered pavement systems.

When a soil mass is subjected to an impulse loading, compression waves are transmitted, and upon contact with a denser material such as a rock layer, these waves are either reflected or refracted. When a brief impulse is produced, the first arrival near the disturbance can be considered to be the compressional wave, and by measuring the arrival time at two different points on the surface the compressional wave velocity can be determined. When a low velocity medium overlies a high velocity medium, it is possible to determine the compressional wave velocity in the second layer. At a distance from the disturbance in such a situation, the first wave front arrival results from the compressional wave that travels through the top layer, in the second layer parallel to the surface and then back through the first layer to the detection point. At a sufficient distance from the disturbance, the arrival of the compressional wave front occurs before the arrival of compressional wave front that is traveling directly through the top layer. Difficulties occur in pavement systems where high velocity layers overlie lower velocity layers. For this case, only the velocity of the higher velocity upper layer can be measured. Since the wave signal travels faster through the dense upper layer, the velocity of the lower layers is not observed.

Impulse vibration testing is, therefore, not applicable to layered pavement systems where strong, high-velocity layers occur at the top and grow progressively weaker with depth. However, this procedure has been used to obtain compression wave velocities of pavement layers during construction. When both compression wave velocities and shear wave velocities are known, Poisson's ratio can be obtained from the formula:

$$\nu = \frac{1-2\left(\frac{Cs}{Cc}\right)^2}{2-2\left(\frac{Cs}{Cc}\right)^2}$$

Resonant Column Method. The apparatus and procedures for the resonant column test used to obtain moduli of soil specimens in the laboratory by means of a vibratory technique were conceived in 1936 (48). In 1950, the apparatus was modified (49) so that a uniform confining pressure could be applied to the sample and thus approximate the in situ stress conditions. The early equipment, which consisted of one test device to determine the shear modulus and another to determine the compression modulus, was later consolidated into one apparatus that would determine both types of moduli (10). In this apparatus, a 1.4-in.-diam by 3.0-in.- high soil specimen was vibrated in either the longi-tudinal or torsional mode by a speaker driver system, and the sample response was monitored by a phonograph needle pickup system. Further modifications in 1962 (11) increased the sample size to 4 in. in diameter by about 15 in. high and used electromagnetic vibrators and piezoelectric accelerometers as the excitation and pickup systems, respectively. The torsional drive system was improved by using two drivers instead of one. Equipment is available with the capability of testing a specimen under a principal stress ratio (PSR) other than 1; i.e., the axial stress on the cylindrical sample could be greater than the radial confining pressure, which was applied by means of nitrogen gas and could be varied from 0 to about 100 psi.

When the resonant column method is used, the sample response is monitored for a range of frequencies in both the longitudinal and torsional modes of excitation to determine the resonant frequency of the soil column. The fundamental resonant frequency of the sample is assumed to be the lowest frequency at which the ratio of the acceleration at the top of the sample to that at the bottom of the sample is a maximum, and the phase angle between the sine wave at the top and bottom of the sample approaches 90 deg. The desired dynamic soil properties can then be found by means of the following relations (10):

$$G = 16 f_t^2 L^2 \rho$$

$$E = 16 f_l^2 L^2 \rho$$

$$C_s = 4 f_t L$$

$$C_l = 4 f_l L$$

$$C_c = V_l \left[\frac{1 - \nu}{(1 + \nu)(1 - 2\nu)} \right]^{1/2}$$

$$\nu = \frac{E}{2G} - 1$$

where:

G = shear modulus

f_t = fundamental torsional resonant frequency

L = length of the sample

ρ = mass density of the sample

E = compression (Young's) modulus

f_l = fundamental longitudinal resonant frequency

C_s = shear wave velocity

C_l = longitudinal wave velocity

C_c = compression wave velocity

ν = Poisson's ratio

The equations given above are simplified versions that consider the elastic case only. Equations are available, however, that consider the soil to be a viscoelastic material and yield a value of the damping property and values of moduli and velocity that account for material damping.

The results obtained from this type of labortory test, like those obtained from any other conventional laboratory tests, are dependent upon the quality and representativeness of the sample and upon the test load conditions. Ideally, it is desirable to have undisturbed samples, but if the samples are remolded, the water content and density should be the same as those of the undisturbed material. Also, the stress conditions for either the undisturbed or remolded sample should be the same as those for the material in situ. Under most circumstances, it is generally impossible to obtain an accurate estimate of the state of stress of either in situ material or remolded specimens. Therefore, in situ residual stresses generally are not reproduced in remolded samples, and the confining pressure for the laboratory specimens can

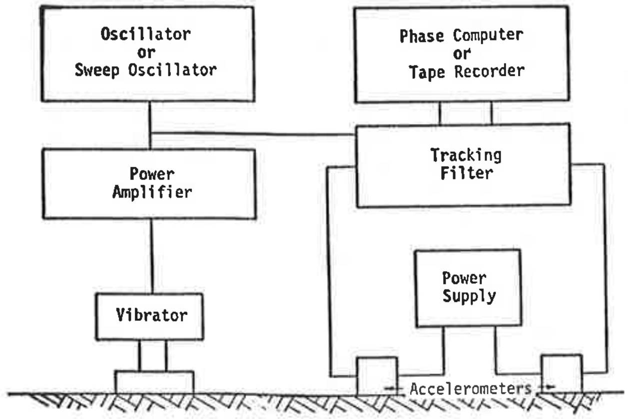


Figure 36. Schematic of wave propagation test setup.

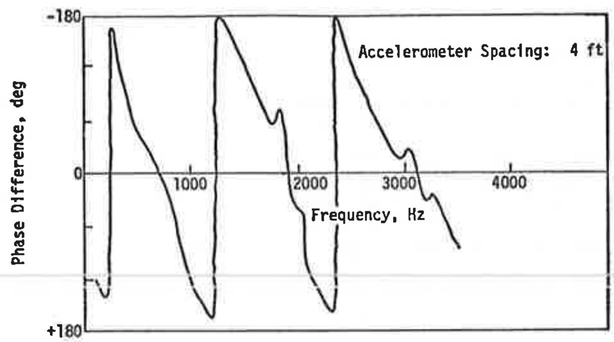


Figure 37. Phase/frequency relationship between 0 and 3500 Hz.

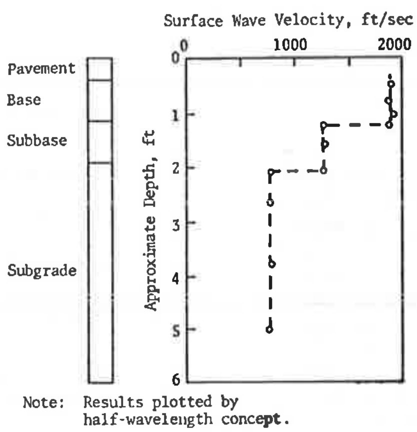


Figure 38. Velocity/depth relationship of AASHO Road Test.

only approximate that for the in situ material. Results of comparison studies (12) on 14 sites showed that laboratory-determined moduli ranged within ±50 percent of in situ moduli. This degree of agreement was considered good, and differences were attributed to variations in confining pressure and density.

Pulse Method. The pulse method is primarily applicable to PCC samples. In this technique, a sequence of pulses (compression waves) is generated at one end of a specimen and the signal is received at the other end of the specimen. The delay time between pulse input and pulse output is used to compute the wave velocity. The time of travel should be smaller than the spacing of pulses in order to prevent interference. Depending on the type of pulse introduced into the specimen, the longitudinal and shear wave velocities can be determined. Application of the pulse method to concrete was reported as early as 1925 (13); the method has been used by several investigators since then. Several investigators (14) have revealed that there could be a relationship between pulse velocity and compressive strength; but, results seem to indicate that (1) there is no unique relationship between the pulse velocity and the concrete strength, (2) limited correlation exists between change in pulse velocity versus change in strength, (3) variation of pulse velocity shows a difference in the concrete strength, and (4) a continuous decrease in pulse velocity with time shows a deterioration of the material with time. Sometimes the pulse velocity method is even used as an acceptance test for structural concrete. In the laboratory, the pulse velocity method has been used to measure the depth of artificially induced cracks.

Data Interpretation

The most difficult aspect of the wave propagation techniques is that of interpretation and analysis of test results. The wave propagation method of testing relies on the ability to interpret the data obtained in the field so that the characteristics of the structure beneath the surface may be determined.

From the theory of elasticity for a homogeneous, elastic half-space, the shear modulus, G, and the compression modulus, E, can be obtained from the shear wave velocity from the relationships:

$$G = \rho C_S^2 \quad \text{and} \quad E = 2(1-\nu)\rho C_S^2$$

where:

ρ = mass density = $\frac{\gamma}{g}$

γ = wet density of material

g = acceleration due to gravity = 32.2 ft/sec/sec

C_S = shear wave velocity

Generally, the measured wave velocity is assumed equal to the shear wave velocity for the above computations.

Empirical Procedures

Early investigators (21) observed that the elastic moduli calculated from wave velocity measurements could be correlated as to material types in a layered pavement system if the effective depth of propagation was taken equal to one-half the wavelength. Fig. 38 shows this relationship to be approximately true for a highway type pavement, but this type of interpretation becomes difficult on heavy airport type pavement as shown in Fig. 39. A theoretical justification (55,56) and adjustment factors to the half-wavelength-depth relationship have been developed; however, these factors appear to be only partially corrective.

Attempts have been made to correlate the wave velocity or the computed modulus values to other known pavement parameters. Early correlations (18) were developed between the compression modulus, E, and the California Bearing Ratio (CBR). Later refinements (19) were made to this correlation, which are shown in Fig. 40. Other correlations (20) have been made between E and plate bearing subgrade modulus, K. However, these correlations were generally fairly poor. Conventional tests such as the CBR and K produce deformations that are not completely recoverable and are, therefore, partly in the plastic range. The rate of loading and magnitude of strain are such that the response of the pavement layers to the wave propagation vibratory tests is in the elastic range. From this, a perfect correlation between wave velocity or E modulus would not be expected.

Tests (21) on full-scale airfield test sections at WES indicated that the wave velocity measured at depth in a pavement structure is a function of the thickness of overlying material. During construction of the test pavements wave velocities were measured from the top of each successive layer. Using the one-half wavelength as the effective depth, the results of a typical test are shown in Fig. 41. The velocity for the subgrade, for example, was quite different when measured with the vibrator on top of the subgrade material as opposed to results with the vibrator on top of the newly constructed pavement. Other pavement sections showed this change in velocity to be a function of the pavement thickness. This series of tests did show apparent changes in the wave velocity with the application of traffic as shown in Fig. 42. Earlier work (22) had shown the wave velocity technique to be capable of measuring increase in the strength of stabilized pavement materials due to the setting-up action of the stabilizing agent.

Theoretical Procedures

Because the relationships between wave velocity and frequency are complex, theoretical interpretation of the test results have been difficult. Generally, the theory (6,23) has been based on the propagation of bibrations from plates (or multiple plates) lying on uniform layers of other materials. For purposes of analysis, test results are expressed as a graph of velocity versus wave length, and then fitted to theoretical curves deducted from the propagation of plane waves in an equivalent type of layered construction. The thickness and elastic properties in the theoretical curves are adjusted to provide the best fit to the experimental results. A recent study (24) of the theoretical analysis of wave propagation phenomena provides solutions to the general differential equations of motion for layered elastic structures obtained for various assumed boundary conditions.

The general idealizations that are made in developing the dispersion relationships are that the propagating media are homogeneous, continuous, isotropic, and linear elastic, and the propagating structure consists of uniform horizontal layers with plane and parallel interfaces , with continuity and displacements across interfaces where applicable. Development of dispersion curves

Figure 39. Velocity versus depth, Randolph AFB, Texas.

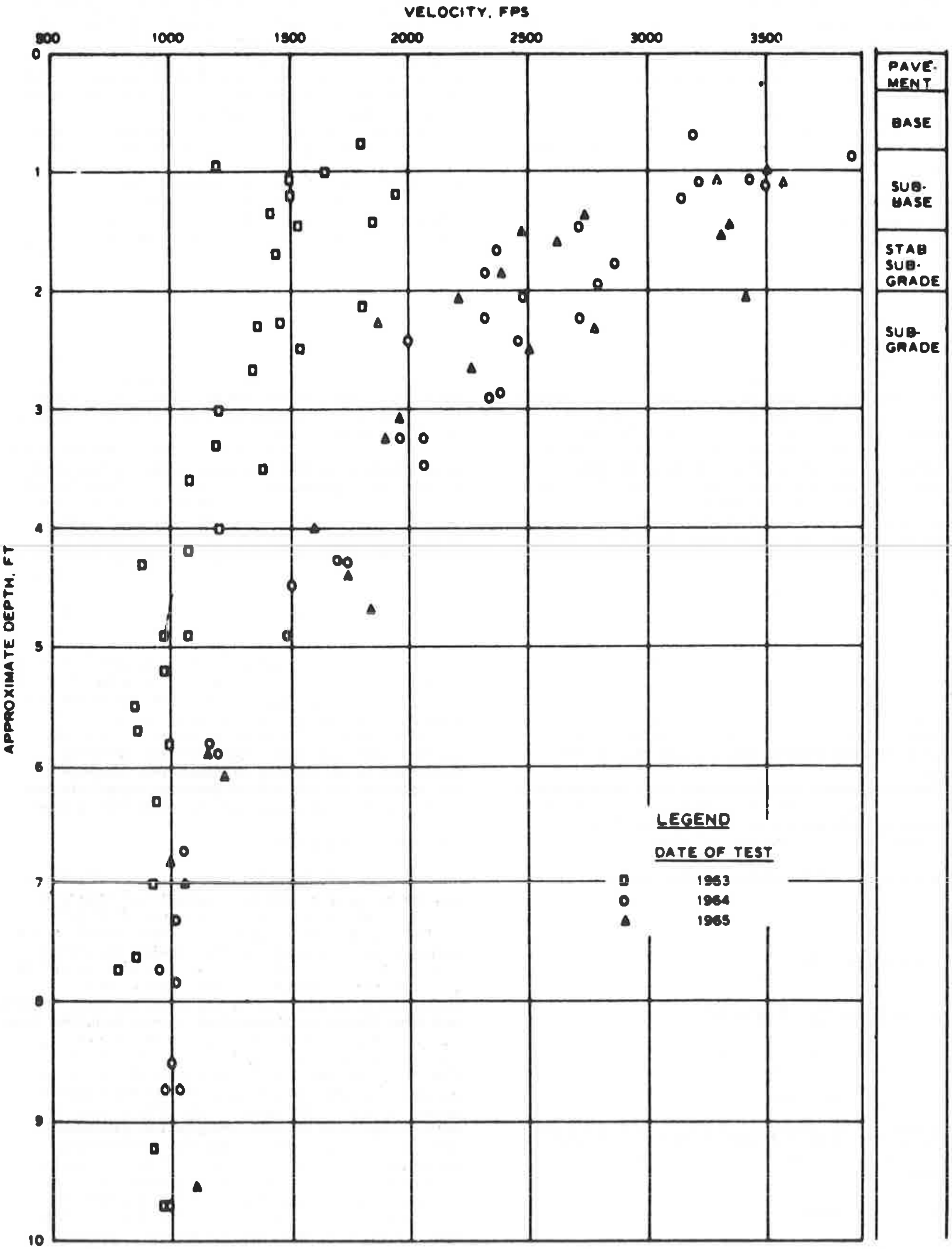


Figure 40. Dynamic E-modulus versus CBR (comparison of Shell and WES correlations).

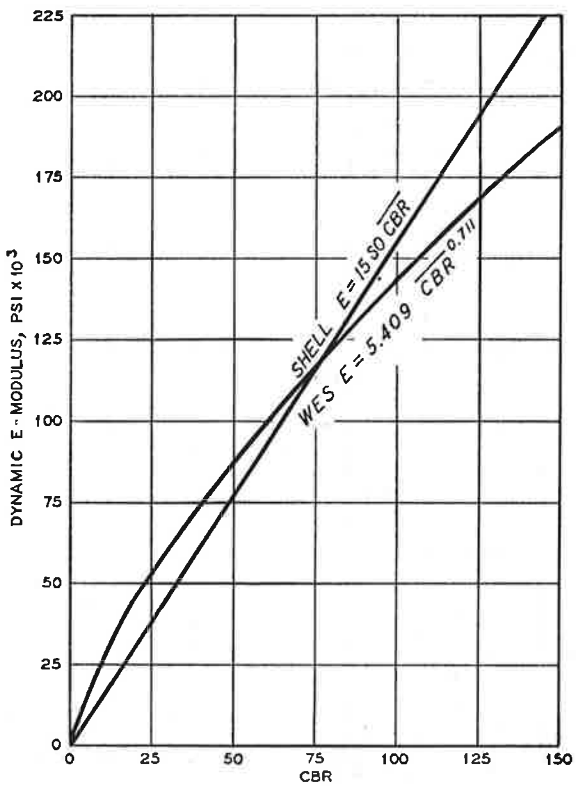


Figure 41. Wave velocities determined during construction of item 4, lane 1, flexible pavement test section.

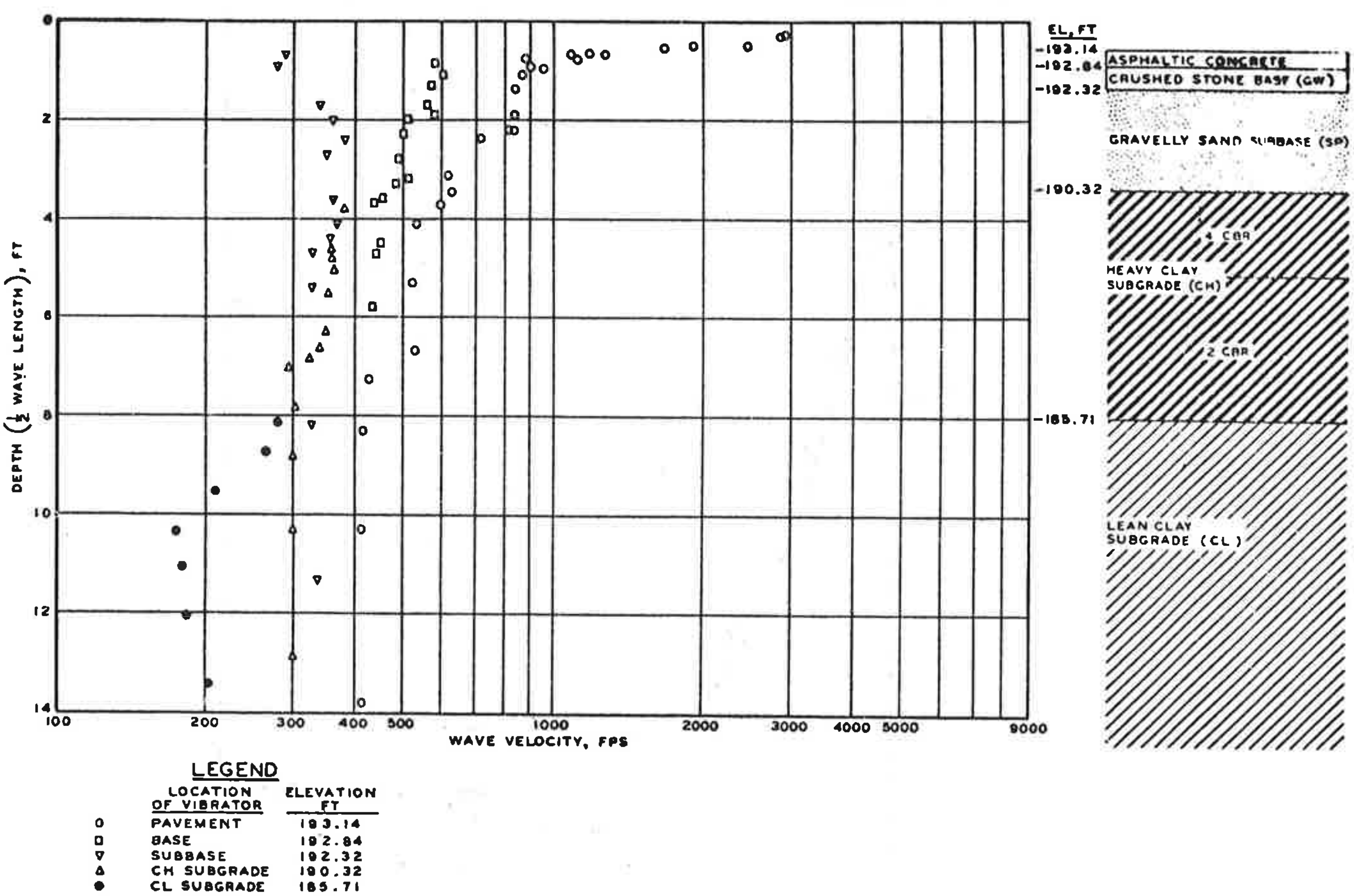


Figure 42. Wave velocities determined during traffic of item 4, lane 1, flexible pavement test section.

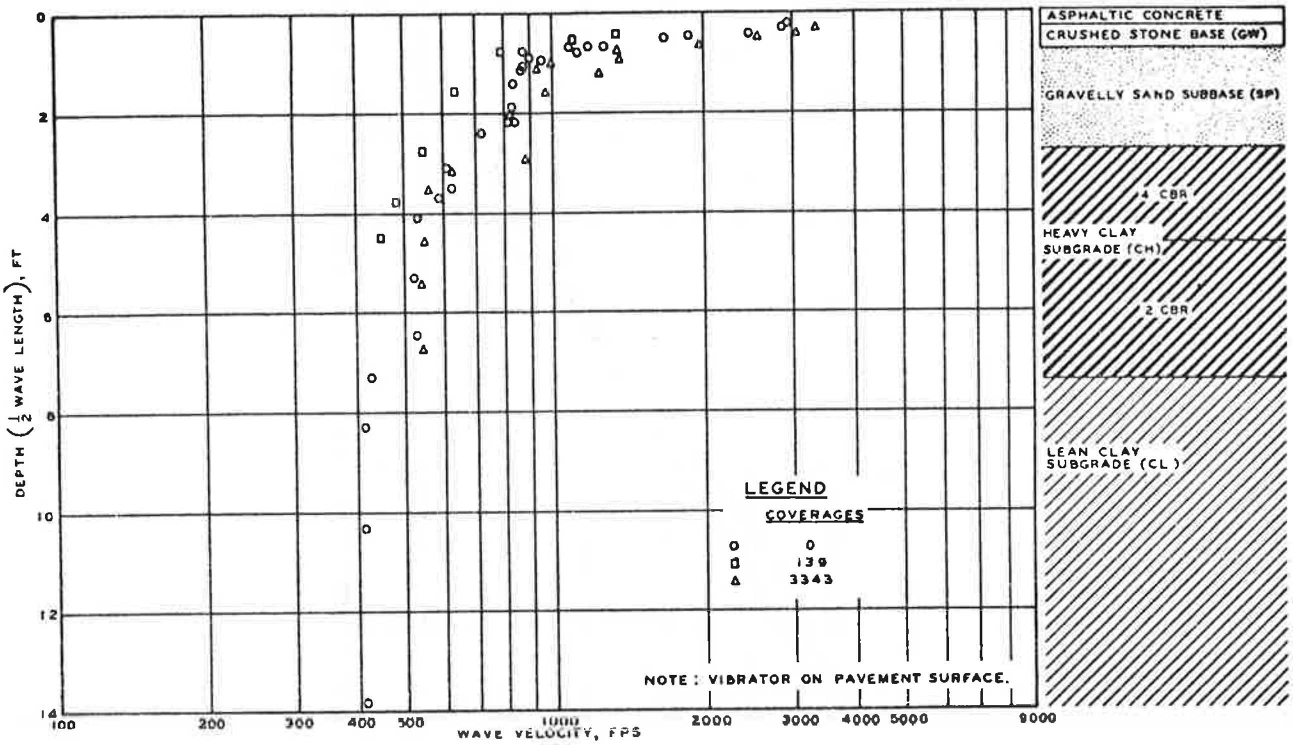
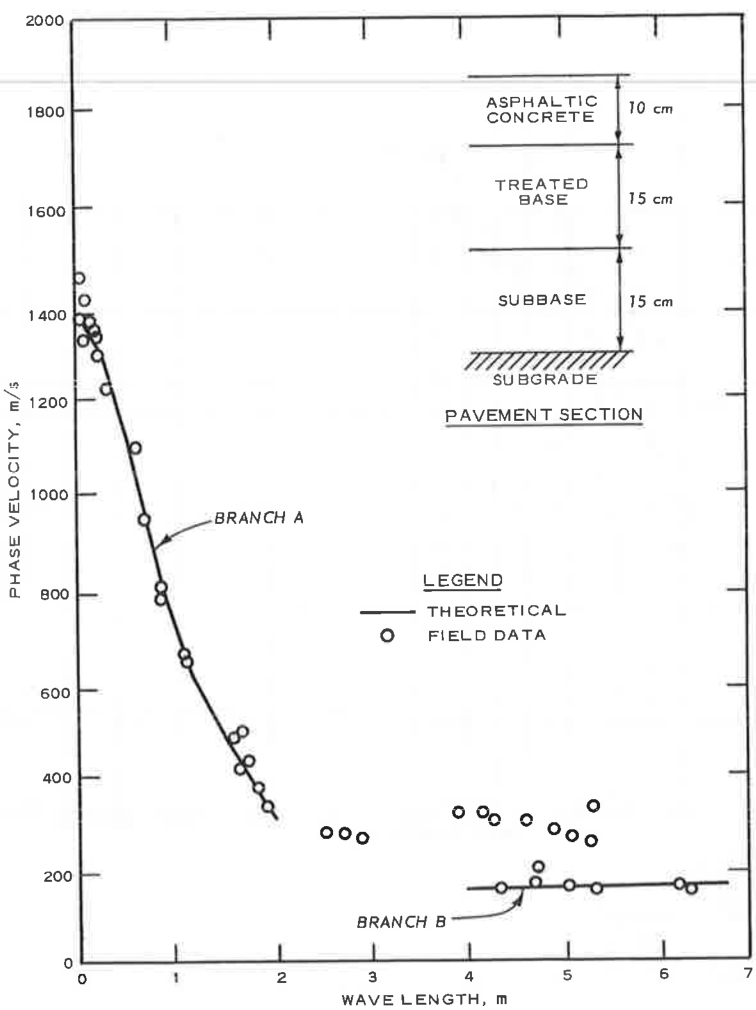


Figure 43. Typical dispersion curves.



generally follow earlier works of Lamb (25) and Jones (26). A theoretical dispersion curve for a three layer structure is shown in Fig. 43.

A dispersion curve for surface elastic waves generally gives the phase velocity as a function of the frequency. Dispersion curves for surface waves in a pavement are obtained by the technique of forced vibration of the pavement surface using a mechanical vibrator. The primary cause of the dispersion of surface waves in a pavement is the layered structure of the pavement system; only a small amount of intrinsic dispersion is associated with the material properties of the pavement layers (27).

Two kinds of surface waves are considered for use in pavement evaluation--Rayleigh waves and SH-waves (28). Rayleigh waves are elastic surface waves resulting from the coupling of P-waves and S-waves that are reflected from the pavement surface; these waves can be generated by using a mechanical device which is operated in a vertical mode of motion on the pavement surface. SH-waves are simply ducted polarized shear waves, i.e., horizontally polarized shear waves which are confined to the layers near the pavement surface (29). SH-waves are produced by operating a mechanical vibrator in a torsional mode of motion on the pavement surface.

Surface waves are essentailly describable as a waveguide phenomenon and, therefore, exhibit the well-known waveguide modes of motion which may be classified as symmetric or antisymmetric (28). In addition, each mode of the dispersion curve may be associated with several branches depending on the relative magnitudes of the phase velocity and the shear wave velocities of the pavement layers. Each branch of the dispersion curve is generally associated with cut-off frequencies in the low- and high-frequency ranges. When present, these cut-off frequencies give the dispersion curve for pavements its characteristic discontinuous appearance. The values of the cut-off frequencies depend on the thicknesses and shear moduli of each pavement layer. Therefore, the in situ measurement of the various branches and cut-off frequencies of the dispersion curve obtained from the technique of forced vibration of pavements is sufficient to determine the elastic modulus and thickness of each pavement layer (30).

Applications

Although much effort has gone into research on the wave propagation methods, only limited use has been made of these procedures for pavement evaluation. Interpretation of the wave velocity test results to obtain the elastic constants of pavement layers is the greatest obstacle. Generally, the dispersion curve technique has been the most satisfactory approach.

The only agency found to be actually using the wave propagation method on a routine basis is the Air Force Weapons Laboratory through the CERF (31). The procedure followed by CERF is to measure the phase difference between two pairs of accelerometers while sweeping frequency between 5 to 3500 Hz. A 1000-lb peak dynamic force is applied with the CERF 5000-lb electrodynamic vibrator. From the data, dispersion curves are developed to identify the elastic modulus and shear modulus of the surface and subgrade materials. Elastic constants of intermediate layers of a multilayered system are presently estimated. The elastic constants for each layer are then entered in the AFPAV code to predict stress and surface deflection for any of 12 aircraft types including multiple-wheel aircraft. A limiting strain is used for the surface layer with work being done on the allowable strain levels for the subgrade. Good agreement has been obtained between predicted and measured surface deflection.

EVALUATION TECHNIQUES UNDER DEVELOPMENT

Light Beam Technology

Application of new laser technology to measurement of surface deflection is currently under preliminary joint development by three Seattle firms (71). It is expected that a basic unit can be attached to the underside of a loaded truck or aircraft to measure deflection caused by moving wheel loads. The procedure would be as follows:

1. The operating speed could be from 0 to 60 mph.
2. Deflection can be measured at any selected interval, say 50 ft.
3. The apparatus places a target at the point of measurement, for example, on a spot in front of the vehicle.
4. The scanning laser locates the target.
5. The primary laser locks onto target and monitors vertical displacement as dual tires move over and past target. Several (5 to 10) measurements are made at each point in order to define maximum deflection and shape of basin.
6. Because light beam measurements are made from a moving platform, the longitudinal profile is also monitored.
7. Data are recorded and analyzed or compiled on-board and are treated statistically as input to management system.

The Air Force Weapons Laboratory (AFWL) is attempting a similar approach through the use of light emitting diodes which produce intense light beams. The AFWL has a workable system and has collected data from moving aircraft (72).

Photo Technology Theory

New techniques in photogrammetry are being developed at the University of Washington using 2 pairs of cameras carried by a loaded, moving vehicle (71). Analysis techniques permit accuracies to 0.001 in. deflection. Data, in the form of photographs, can be analyzed to varying degrees (vertical deflection at a point deflection basin in x and y directions or contoured, or continuous longitudinal) or an automated computerized stereoplotter at a central location.

Energy Transfer

This appraoch consists of the concept that there is a functional relationship between the cumulative energy as measured by cumulative peak deflections imparted to a given pavement system and the condition of that system (73,74).

The concept was tested by applying it to load-deflection and performance trend data gathered in the AASHO Road Test. Regression analysis was performed to find a relationship that predicted the level of the Present Serviceability Index (PSI) as a function of the pavement profile and a measure of the cumulative energy imparted to the pavement.

Because of the paucity of airfield condition and deflection data, indirect means were used to test the working hypothesis for airfield pavements. Traffic records and construction histories for two Air Force Bases were analyzed. The analysis indicated there is a threshold cumulative total peak deflection at which cracking develops in airfield pavements.

Field testing provided insight into the effect of the thickness of overlays on the energy imparted to a pavement and led to the development of a procedure to provide the thickness of an overlay so that the pavement will perform satisfactorily under an anticipated traffic volume.

It was concluded that, in the future, performance trends in airfield and highway pavements can be predicted from knowledge of cumulative total peak deflections.

A means of developing the loaded peak deflection basin is proposed based on the hypothesis that time-dependent functions could be calculated from a pavement's dynamic input and output deflection responses; that these functions could be obtained, in a mathematical sense, without the need to simulate respective material performance to determine values for pre-selected descriptors, and that these time-dependent functions could be employed to predict deflection output responses when the pavement was subjected to any imposed load.

The pavements response to imposed loads can be generated by use of the pavements transfer function developed from loading with a common prime mover. The use of these time-dependent transfer functions is associated with the development of characteristic vehicular laodings.

Dynamic Deflections Response Theory

A theoretical approach to airfield evaluation is currently being pursued by the FAA through research at WES (31). The dynamic deflection pavement response is used with dynamic theory to predict elastic constants of pavement layers. These predicted constants are then to be used with layered analyses and limiting strain criteria to predict allowable loadings and aircraft operations on airport pavements. The theoretical analysis incorporates a nonlinear load-deflection response model with the frequency response of pavements to vibratory loadings to produce the elastic constants for the pavement layers. In its present form it is necessary to assign elastic constants to the upper pavement layers based on conventional soil properties. The trial procedure is expected to contain nondestructive techniques for measuring all of these constants.

SUMMARY

Several approaches are available for nondestructive evaluation of highway and airport pavements. These include static deflection measurements using full scale loads, dynamic deflections using steady state or impact loads and the analysis of wave propagation measurements for determining the properties of individual layers. All of the techniques have certain merits, although some have found broader acceptance than others. Since the properties of the layers within pavements are influenced by variable environmental factors like temperature and moisture, nondestructive measurements are significantly influenced by these factors.

Due primarily to their simplicity and ease of testing, static deflection devices have found wide application. Through use of limiting deflection criteria, static deflections are commonly used to rate existing load capacities and to predict overlay requirements.

Because steady state deflections correlate well with static measurements, they are often used as a rapid means of obtaining data for use in static based procedures. In addition they have been employed to correlate directly with allowable loadings determined by conventional thickness design procedures. Much is yet to be learned about the relationship between the force generator static weight, dynamic force and loading frequency. This is particularly true for low frequencies where it becomes difficult to produce large sinusoidally varying forces and inertial motion sensors have very low output. Steady state deflection measurements represent recoverable deformations of the entire pavement structure and do not give any information about the plastic (or nonrecoverable) response of pavement structure nor do they provide direct measurements of the individual pavement layers.

Although impact testing techniques have not, as yet found wide acceptance, this method of testing offers an extremely rapid technique for determining the dynamic load response characteristics of pavement structures. In order to accurately define the significant frequency response characteristics through Fourier analysis, precise definition of both the impulsive force as well as the deflection response is required. In addition the peak of the force impulse divided by the peak of the deflection response can be taken as a measure of the overall pavement stiffness. Such a stiffness measurement, more or less represents an average for the frequency spectrum contained in the force impulse. The shorter the duration of the force impulse the wider is the frequency range which is represented. Impact testing like static and steady state testing characterize the entire pavement structure and does not provide direct evaluation of the individual pavement layers.

A great deal of effort has been devoted to the development of the wave propagation technique because it offers promise of providing elastic moduli values for all of the individual pavement layers. The technique has met with only limited success. Some recent procedures are finding application in pavement evaluation by inferring from wave propagation data, the elastic moduli for surface and subgrade layers and estimating the properties of intermediate layers. The main difficulty with this technique continues to be on the interpretation of the wave velocity measurements and no generally applicable interpretation scheme has yet been devised for multi-layed structures.

Research on new techniques appear useful. Laser devices, light emitting diodes, and photography techniques offer a potential for rapid deflection measurements using full-scale moving loads. The deflection results would, of course, suffer some of the same limitations as do static deflection measurements.

The large variety of test devices and techniques being used in the nondestructive testing of pavements, and the large amount of literature reported on the subject, indicate both the tremendous interest in pavement evaluation, as well as the need for further coordination between researchers. There still remains much to be learned about the various aspects of nondestructive test equipment, data interpretation, and application of results into useful information and guidance to pavement equipment.

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