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MENT EVALUATION:
PHASE I. PAVEMENT EVALUATION EQUIPMENT

March 1975

Final Report

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16. Abstract The purpose of this study was to determine the most promising kinds of equipment to develop for the purpose of evaluating the structural condition of pavements. All areas of technology both in the highway field and in fields outside of highway technology were to be studied to determine the most promising types of equipment.

In order to decide what needed to be measured in determining the structural condition of a pavement, all 50 states, some Canadian provinces, and some city and county agencies were asked to name the factors they consider to be important in their assessment of pavement condition, and state how heavily they weight each one. Results of this survey of current pavement condition rating methods are included in this report. The two most important variables were found to be stiffness and cracking.

Highway equipment reviewed include static deflection, steady-state vibration, impulse, and wave propagation techniques. Non-highway equipment include acoustic holography, infrared microscope, vibroseis, photo diode scanners, microwave sensing, and the duomorph. Several new types of measurement and analysis schemes were proposed.

Ten pieces of equipment were recommended for future development, two of which were to be developed in Phase II of this study: the duomorph and the mobile acoustic crack detector.

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PAVEMENT EVALUATION

CHAPTER I

INTRODUCTION

Methods of pavement evaluation can vary all the way from one individual judging in a subjective manner what he "feels" should be done to maintain quality pavement on a section of roadway - to the use of detailed mechanical and visual procedures. Both methods serve the same goal but the results of each should not be expected to be necessarily the same. Consequently, some standardized way of describing pavement evaluation is desireable.

The two most widely accepted categories of pavement evaluation are (1):

1. Serviceability - performance studies of functional pavement behavior, and
2. Mechanistic evaluation of structural adequacy.

The serviceability - performance studies are generally considered to be user oriented and thus indicate how the pavement performs as a riding surface. Although there are many indicators which could be examined, roughness and skid measurements are often used (2). A mechanistic evaluation of the pavement structure measures items such as structural capacity, distress, roughness, skid resistance, and more (3). This type of evaluation can be accomplished at least in part by use of a condition survey.

A good example of the differences between the two categories of pavement evaluation has been presented by Haas (3). If a crack forms in the pavement surface today, it will have no immediate effect on the riding quality of the pavement. Although this new crack is not accounted for by serviceability measurements, a mechanistic evaluation can identify and measure this item of distress. Such a crack can be expected to be an indication of future serviceability decline. Thus, a decrease in the serviceability of a pavement is preceded or paralleled by a deterioration in its structural condition.

Since the AASHO Road Test, the serviceability - performance concept and ensuing evaluation techniques have advanced significantly and will probably continue to do so. But, the mechanistic type of evaluation for estimating the structural condition of a pavement is still the most arbitrary and subjective evaluation category. As such, this area appears to hold the most promise for immediate developmental work.

STRUCTURAL CONDITION SURVEYS

There are two major purposes for evaluating the structural condition of pavements: to furnish information for design and to provide data for rehabilitation decisions. The first of these purposes requires detailed information either for the design of new construction or for determining the amount of rehabilitation (e.g. overlays, seal coats, reconstruction) that will be required. Data for the second of these purposes, rehabilitation decisions, will not need to be so detailed but they should be consistent in ranking the distressed condition of all pavement sections in a highway network.

The two main purposes of structural condition evaluation require two kinds of condition survey: a detailed survey for design data and a rapid survey for decision data. Because each of these surveys has its own unique objectives, it is not surprising that each should also have its own criteria for what determines an acceptable survey. In general, the decision--or rapid--survey is interested in a quick but comprehensive view of everything that is going wrong with the pavement. It is primarily interested in the distressed condition of a whole section of pavement and its objectives are better served if its assessments of the level of distress are consistent from one section to the next.

As such, the "decision survey" forms only a part of what is called the "sufficiency survey" in current highway practice. A sufficiency survey also considers factors such as geometry, traffic and obstructions (4).

The design--or detailed--survey is concerned with the structural adequacy of the pavement section to carry future anticipated loads. It is an attempt to gather data such as thickness, stiffness of the layers, their material properties, and crack spacing to determine the thickness of planned overlays or the depth of a pavement to be reconstructed. Seal coats are rarely placed to increase the structural stiffness of a pavement. Rather, they are intended to plug the cracks and reduce the rate of deterioration.

The "design survey" is intended to be broader in scope than the "structural evaluation" commonly used in current highway practice. The structural evaluation is usually associated with deflections and pavement layer moduli. In addition to these data, a design survey may gather other kinds of data such as crack spacing, which is required in some overlay design procedures (5).

Ideally, there should be some correlation between the results of these two surveys: the decision survey should indicate reliably which sections need work and the design survey should tell how much work is needed.

The common tie between these two survey systems is distress: the greater the distress, the more urgently the observor feels the pavement needs attention. The purpose of rehabilitation design is to halt or retard the appearance of various signs of distress and hopefully to return the pavement to its original strength.

Rapid or Decision Surveys

There are several forms of pavement distress, each of which provides the designer with its own clues for diagnosis. A partial listing of the kinds of distress used by various state agencies to evaluate the condition of a pavement is given in Table 1.1. It is beyond the scope of this report to consider which of these signs of distress is most important in determining what maintenance should be done.

Instrumentation to measure the severity of these important signs of distress consistently and rapidly can be very useful to the decision process. It was for this purpose that a careful review was made of existing state and some county pavement condition rating systems. The results of this study are presented in Chapter 2 of this report and a summary of the rating systems is included as Appendix A. It will be no surprise that various forms of cracking consistently emerged as the dominant indicator of distress requiring maintenance.

As a result of this study, the conclusion was reached that the visual condition survey can never be replaced. However, it can and should be supplemented by auxiliary equipment to measure the critical variables in a consistent manner. The critical variables will always include ride, cracking, deflection, and skid, each of which can become important enough by itself to initiate some form of rehabilitation action. The only one of these which is not measured by some instrument in standard highway technology is cracking.

A greater interest in cracking of all sorts is evident in this study of pavement rating systems. It appears to be the most reliable or most consistent indicator of the remaining life of the pavement. Cracking indicates distress due to either traffic or climatic factors or both. Once cracking appears, the factors which influenced its appearance will continue to operate

TABLE 1:1

PAVEMENT DISTRESS INDICATORS

Flexible Pavement	Rigid Pavement
Transverse cracking	<u>ALL RIGID PAVEMENTS</u>
Longitudinal cracking	Roughness (or Ride)
Multiple Cracking (beginning of alligator cracking)	Surface Deterioration -Raveling -Scaling
Alligator cracking	Spalling
Rutting	Longitudinal cracking
Raveling	Patching
Patching	Faulting
Flushing (or bleeding)	Pumping
Corrugations	Failures per mile
Roughness (or Ride)	Blowups
	<u>CONTINUOUSLY REINFORCED CONCRETE</u>
	Crack Spacing
	% Intersecting Cracks
	<u>JOINTED CONCRETE</u>
	Spalled Joints
	Faulted Joints
	Cracked Panels
	Broken Panels
	Transverse Cracking

and accelerate the deterioration of the pavement condition. The more extensive or severe the cracking, the sooner some form of rehabilitation must be done. Rehabilitation and betterment programs are planned and budgeted and are more effective when reliable predictions can be made of the remaining life of each pavement section in the roadway network. Traffic fatigue, thermal fatigue, and shrinkage cracking can all be predicted at the present time by various methods ranging from empirical to experimental or theoretical. Each method can predict the growth of the size and extent of cracking once the current condition of cracking is known. The detection of cracks that are small or as yet unseen will give an earlier warning of the need for maintenance.

Consequently, an attempt has been made in this project to examine the effectiveness of several methods of detecting visible and invisible cracking both by visual observation and by non-destructive detection using various kinds of equipment that are not currently used in highway-oriented fields.

Detailed or Design Surveys

Once the decision has been made to perform some kind of maintenance on a given section of pavement then what is usually done is to obtain detailed information of the structural adequacy of the section. Various methods are used in determining the structural soundness of a pavement section, the most fundamental being related to a Benkelman beam deflection. Most pavement design experience up to the present time has been related in some way or another to this kind of deflection. The efficiency of most non-destructive testing apparatus is eventually determined by some sort of comparison with Benkelman beam data. This is a very practical consideration for without this correlation much of the useful pavement design and performance experience of the past several decades would have to be abandoned. Consequently, all non-destructive equipment, to be useful, must be able to produce measured results which can reproduce a pavement surface deflection. The two major non-destructive techniques are vibratory: one measures surface deflections and the other measures wave propagation velocities. The means of empirically correlating between the deflection methods and Benkelman beam results is apparent. Wave propagation methods are more indirect. They must first determine the elastic properties of the layers and then use elastic theory to predict Benkelman beam deflections. A similar analytical procedure may be followed using surface deflection data. In either case, the analysis methods that are currently available for either surface deflection or wave propagation techniques cannot produce reliable values of elastic moduli for more than two layers without a considerable amount of hand manipulation, assumption, and costly computer iteration.

Current developments in pavement design methodology show that there is a distinct advantage to choosing a non-destructive testing method which can produce reliable field values of elastic modulus. Most research and experimentation with new materials and new combinations of traditional paving materials and most of the recent theoretical development of pavement design methods has been based explicitly on elastic theory. The ultimate objective of using this theory is to unify the various approaches to pavement design and permit an optimum use of materials while maintaining a minimum total cost of the pavement. Selection of pavement evaluation equipment for design purposes should consider carefully how it will be used in the future when these current developments in design methods are implemented into design practice. The ideal piece of equipment for this transition period is one which produces a deflection for use in current design methods and whose output can be analyzed conveniently to produce material properites for us in future design developments.

Underlying the assumption that design methods based on elastic theory will eventually be widely adopted are several important facts:

1. A number of current design procedures in use by state highway departments and others recommended by material suppliers are already based on elastic theory.
2. Several important workshops and symposia have arrived at the conclusion that fruitful developments in pavement design would be in the area of elastic theory (e.g. Ref. 6).
3. Several studies of the results of full scale field tests at the U.S. Army Corps of Engineers Waterways Experiment Station (Ref. 7) and the AASHO Road Test (Ref. 8) and in South Africa (Ref. 9) have shown that even for heavy loads, pavement materials have a reasonably linear load-deflection characteristic, the principle of superposition appears to hold, and elastic theory is likely to give a sensibly accurate representation of a layered pavement structure. Non-linearities do occur, of course, but field tests indicate that they occur when the subgrade soils are saturated with the spring break-up or when one of the layers is cracked into discrete, isolated slabs.

It is certain that if the assumption of linearity is not at least reasonably accurate then there is little point in devising elaborate analysis schemes to extract elastic moduli from non-destructive test data. If non-linearity is the rule, then what will be required for pavement design evaluation will be some form of destructive evaluation, lab testing, hand manipulation of calculations, and some kind of assumption on the <u>form</u> of the non-linearity which cannot, in any event, be verified by independent means. If linearity does not hold at least approximately true even for heavy loads, then it may be a more efficient use of pavement evaluation equipment to be concentrated upon developing more empirical design relations. The technical approach of this report and its recommendations for equipment development are predicated on this important assumption of linearity and the expectation that linear elastic theory will become more widely used as a basis for pavement design.

Pavement Evaluation Equipment Criteria

Although these criteria will be discussed more in detail in subsequent chapters, it is worthwhile to list them here at the beginning of the report.

The purposes of the two kinds of survey are different and, as a consequence, so are the criteria which must be met by equipment used in collecting their data. The decision--or rapid--survey requires speed of operation, repeatability of the measurement, consistency of measurement between sections, and in some cases, it requires some means of correlating its measurements with empirical Benkelman beam data. The design--or detailed--survey requires repeatability of the measurement, an ability of the measurements to be analyzed to determine material properties, and the applicability of the results to present and future design procedures.

SUMMARY

Two kinds of pavement condition survey will always be required: one, a rapid survey for making decisions on required maintenance and the other, a detailed survey for collecting design data. The major variables in each survey will be roughness, cracking, skid, and structural capacity each of which must be measured accurately and consistently so that both decisions and designs may be as cost-effective as possible. Equipment which measures these major variables in a way that can be related both to past design experience and future design methods will be the most valuable to develop.

CHAPTER II

PAVEMENT CONDITION RATING SYSTEMS

The pavement condition rating systems currently in use in the majority of states are a source of valuable information on the types of distress that are critical indicators of the need for rehabilitation. Although the weighting factors applied to the various forms of distress are subjective, they are the result of a careful weighing process by men who are faced with the practical problems of maintaining a high level of service on the nation's highway system.

Many of the rating systems have been revised and updated with continued usage as additional information becomes available. The weighting factors used in these rating systems for various forms of distress have been re-evaluated in the light of experience with the system itself and because of this, they are even more valuable indicators of the critical forms of pavement distress.

SURVEY OF PAVEMENT CONDITION RATING SYSTEMS

Rating system summaries for several state, county, and city agencies were on hand from previous research. To supplement these, a letter was sent to the highway departments in the remaining states, territories, and selected Canadian provinces requesting information on their pavement rating system currently in use or projected for use in the immediate future. A sample of the letter used in the survey is shown in Appendix A along with summaries of the rating systems received.

A review of the replies show that many states use sufficiency ratings or similarily derived methods. These types of systems can include factors other than the structural condition of the pavement and are generally considered to be related to user response. Since the primary purpose of this study is concerned with pavement structural condition, these user response factors are not considered in what follows. However, the rating system summaries contained in Appendix A show how such factors are included in the various overall rating schemes.

Overview of Rating System Study

Out of 58 separate agencies contacted, 44 responses were received or had previously been made available. The agencies included not only the states and selected Canadian provinces but also one county in the state of Washington and two city agencies in Texas.

Most of the agencies contacted responded by furnishing extensive information on their rating methods. However, some agencies cannot be treated adequately due to one of the following reasons: sufficient information was not sent to permit a complete examination of the rating system, development efforts were underway for a new system, or the agency did not reply to our questionaire. Consequently, whatever information provided was used to the greatest extent possible.

Some of the general items derived from the replies are:

1. Number of agencies using or adopting rating systems: 34
2. Number of agencies using a composite numerical rating score: 24
3. Number of agencies using ratings or rating scores in maintenance decisions: 20
4. Number of agencies using rating systems for flexible pavements: 30
5. Number of agencies using rating systems for rigid pavements: 18

The above information probably represents the minimum number of agencies listed for each item.

The replies also indicated that changes can be expected in the future rating systems for some of the state agencies. At least 9 states currently plan changes to their existing systems or will use newly developed ones.

Characteristics of Pavement Condition Rating Systems

The pavement condition rating methods reviewed represent valuable experience in determining the most important kinds of distress. Consequently, they were analyzed in detail from the following three points of view:

1. What are some of the basic components of the various agencies pavement condition rating systems?
2. What percent of the pavement condition rating is determined by distress factors, as opposed to traffic, safety, skid, geometry, obstructions, and other non-distress factors?
3. What percentage of the condition rating is determined by each form of distress such as cracking, rutting, raveling, patching, and so on?

The first question is an attempt to determine the similarities between the pavement condition rating systems in use. The second question is to show how important the maintaining agencies consider distress. Because over half of them use their rating scores in making maintenance decisions, the percent of these scores taken up by distress is an indication of how important it is to measure distress carefully. The third question is to determine what forms of distress are considered most important in these rating scores. The most important forms of distress must be measured carefully for they are the determining factors in rather extensive maintenance and rehabilitation programs. If new equipment is to be proposed for evaluating the condition of a pavement, it should be aimed at measuring these kinds of distress.

Basic Components - Table 2.1 is a summary of the 22 agencies for which the salient features of the rating systems could be shown and is a partial answer to the first question. There are several important similarities in these systems:

1. Over 70 percent of the agencies using numerical rating scores currently or in the near future use their condition rating system in making maintenance decisions.
2. All of the 22 agencies listed have condition rating systems for flexible pavements and at least 60 percent have systems for rigid pavements.
3. The car ride meter (Mays, Cox, PCA, etc.) is used by half of the agencies listed in the table for determining roughness. The Dynaflect and various skid devices are used to a lesser extent.
4. Annual inspection frequencies appear to be the most popular.
5. Generally, the overall numerical rating ranges from 100 (best pavements) to 0 (poorest pavements).

Distress versus Non-Distress - The maximum percentage that distress factors influence the pavement rating score for each agency is shown in Table 2.2. Of the 24 agencies using numerical ratings, only 18 can be listed due to available data.

The percentages range from 17 percent (Arizona) to 100 percent (Maine). No geographical pattern is evident from the distribution of the percentages. On the average, 49 percent of the rating score for flexible pavements and 40 percent for rigid pavements is accounted for by distress factors. Since the remaining percentages account for such items as roughness, traffic, geometry, etc., it is readily apparent that distress considerations are a

TABLE 2.1

SUMMARY OF AGENCIES USING NUMERICAL PAVEMENT RATING SCORES

	Does the agency use the rating score in maintenance decisions?	Does the agency use a rating system for flexible pavements?	Does the agency use a rating system for rigid pavements?	Rating Team Composition	Measuring Equipment Used	Rating Frequency	Numerical Rating Range		Is the numerical rating adjusted for traffic?
Arizona	Yes	Yes	Yes	Unk	Unk	Every two years	100	0	Yes
California	Yes	Yes	Yes	5 to 6 Teams, 2 men ea.	Cox Meter	Every two years	0	176	No
Florida	Yes	Yes	No	5 Teams, 2 men ea.	Mays Meter	Annually	100	0	Yes
Georgia	Unk	Yes	No	Unk	Wisc. Roadmeter Oynaflect Skid	Unk	0	·00	Yes
Indiana	No	Yes	Yes	2 Teams ea. Oistrict, 2 men ea.	None	Annually	100	0	Yes
Kansas	Yes	Yes	Yes	One man	Roughometer	Unk	100	0	No
Louisiana	Yes	Yes	Yes	Unk	Mays Meter Skid	Unk	100	14	Yes
Maine	Unk	Yes	Unk	Unk	None	Unk	5	1	No
Maryland	Unk	Yes	Yes	One man	None	Annually	100	0	No
Minnesota	Yes	Yes	Yes	One team ea. Oistrict, 2 men ea.	PCA Meter	Annually	5	0	No
Nebraska	Yes	Yes	Yes	Unk	Neb. Roadmeter Dynaflect Skid	Every two years	100	0	Yes
New Mexico	Yes	Yes	Yes	One man	None	Annually	100	0	Yes
North Dakota	Yes	Yes	No	Unk	None	Unk	0	49	Unk
Tennessee	Yes	Yes	Yes	One team rates entire state	None	Annually	100	0	No
Texas	Yes	Yes	Yes	One team ea. Oistrict, 2 men ea.	Mays Meter	Annually	100	0	No
Virginia	Unk	Yes	Yes	Unk	Unk	Unk	Unk		Unk
Washington	Yes	Yes	Yes	Four teams, 2 men ea.	PCA Meter	Every two years	100	0	No
*King County, Washington	Yes	Yes	No	Unk	Cox Meter Benk. Beam Skid	Annually	160	0	No
Oregon	Yes	Yes	Unk	Unk	PCA Meter	Unk	Unk		Yes
Utah	Yes	Yes	Unk	2 Teams, 2 Men ea.	Cox Meter Oynaflect Skid	Annually	NA**		NA
Ontario	Unk	Yes	Unk	Unk	None	Unk	100	0	Unk
Corpus Christi, Texas	Yes	Yes	Unk	Unk	None	Annually	100	60	Unk

* This system is under consideration for adoption.
**Not applicable to date.

TABLE 2.2

MAXIMUM PERCENT DISTRESS FACTORS INFLUENCE PAVEMENT RATING FOR EACH AGENCY*

		Flexible Pavements	Rigid Pavements
1.	Arizona	17.0	17.0
2.	California	73.2	--
3.	Florida	50.0	--
4.	Georgia	37.5	--
5.	Indiana	22.0	22.0
6.	Kansas	44.0	50.0
7.	Louisiana	30.0	30.0
8.	Maine	100.0	--
9.	Maryland	40.0	40.0
10.	Minnesota	50.0	50.0
11.	Nebraska	40.0	0.0
12.	New Mexico	40.0	40.0
13.	North Dakota	75.5	--
14.	Tennessee	50.0	50.0
15.	Texas	80.4	88.5
16.	Virginia	48.0	42.0
17.	Washington	50.0	50.0
18.	King County, Washington	37.5	--

*In general, the table does not utilize distress measured by ride meters in the computation of percentages.

significant, though highly variable, part of the individual rating systems.

Importance of Various Kinds of Distress - Table 2.3 shows the percentage of the pavement rating score that is represented by each distress factor and is a further categorization of the information shown in Table 2.2. The distress factors listed are self-explanatory with the exception of the "General" and "Cracking" categories. The "General" category is used to group those forms of distress listed by the various agencies under generalized headings like "structural adequacy". "Cracking" represents all types of cracking listed by a given agency.

The amount that individual distress factors influence the overall rating can be examined in two ways. First, by determining an overall average percentage for only those agencies which use the factor and secondly, by averaging over all agencies. The latter is considered the most informative, because if an agency does not include a given factor, that is an indication that the factor is considered unimportant. The averages in this case were determined for 18 flexible pavement rating systems and 12 rigid systems.

Based on the latter averaging procedure, the "General" category accounts for an average of 13 percent (flexible) and 17 percent (rigid) of the overall pavement rating score. Of all of the specific types of distress, cracking is the most heavily weighted with 17 percent (flexible) and 7 percent (rigid). The next most important forms of distress for flexible pavements are rutting (5%) and patching (3%). For rigid pavements, the next most important forms of distress are spalling (5%) and faulting (3%). Deflections average 3 percent for flexible pavements and are presented for informational value only. They are not considered as distress in this analysis.

Due to the makeup of the data, the percentages shown are approximate

TABLE 2.3

MAXIMUM PERCENT INDIVIDUAL DISTRESS FACTORS INFLUENCE PAVEMENT RATING FOR EACH AGENCY

Flexible Pavements

	General	Cracking	Rutting	Raveling	Deflection	Corrugations	Patching	Pitting	Distortion	Shoving	Pot holes	Flushing
Arizona	17.0											
California		41.0	11.7	14.6			5.9					
Florida		30.0	13.3				6.7					
Georgia					37.5							
Indiana	22.0											
Kansas	4.0	18.0	12.0				10.0					
Louisiana	30.0											
Maine	9.0	44.0	14.0			5.0		14.0	14.0			
Maryland	40.0											
Minnesota		27.5	7.5				15.0					
Nebraska	20.0				20.0							
New Mexico	40.0											
North Dakota		49.0	10.2	4.1						6.1	6.1	
Tennessee	>50											
Texas		27.4	5.9	7.8		7.8	7.8				15.7	7.8
Virginia		35.0	5.0	1.0					2.0			5.0
Washington		18.1	11.1			2.7	4.2		11.1		2.7	
King County, Washington		12.5	6.3	6.3		3.1			3.1	3.1		3.1
Avg. Among Agencies Using the Distress Factor	25.8	30.2	9.7	6.8	28.8	4.6	8.3	14.0	7.6	4.6	8.2	5.3
Avg. Among Agencies Using Flexible (18) or Rigid (12) Rating Systems	12.9	16.8	5.4	1.9	3.2	1.0	2.8	0.8	1.7	0.5	1.4	0.9

Rigid Pavements

	General	Cracking	Faulting	Patching	Blow ups	Spalling	Undulations	Scaling	Pumping
Arizona	17.0								
California									
Florida									
Georgia									
Indiana	22.0								
Kansas	4.0	24.0	4.0			4.0	10.0	4.0	
Louisiana	30.0								
Maine									
Maryland	40.0								
Minnesota		15.0	5.0	15.0		12.5		2.5	
Nebraska									
New Mexico	40.0								
North Dakota									
Tennessee	>50								
Texas		24.1	9.2	4.6	9.2	13.8		13.8	13.8
Virginia		7.0	10.0			25.0			
Washington		9.8	5.9	2.9	2.9	9.8		9.8	8.8
King County, Washington									
Avg. Among Agencies Using the Distress Factor	29.0	16.0	6.8	7.5	6.0	13.0	10.0	7.5	11.3
Avg. Among Agencies Using Flexible (18) or Rigid (12) Rating Systems	16.9	6.7	2.8	1.9	1.0	5.4	0.8	2.5	1.9

and should be considered only as indicating weighting trends for the various distress factors. The most significant trend resulting from this study is that cracking is the single most heavily weighted distress variable used in making maintenance and rehabilitation decisions. Deflections, roughness, and skid number are being measured at present but the most heavily weighted distress factor (cracking) is not being measured by mechanical devices or instruments. Visual methods are the main techniques used and will probably continue to be until new instruments or methods become available.

Equipment Used In Rating Systems

Of the agencies for which information was available, a total of sixteen either currently use or plan to use mechanical devices to assist in obtaining pavement ratings. The devices being used and the number of agencies using them are:

1. Roughness measuring devices (PCA Roadmeter, etc): 16
2. Skid measuring devices: 8
3. Deflections measured by the Dynaflect: 3
4. Deflections measured by the Benkelman Beam: 1

These types and amounts of mechanical devices are used for decision (rapid) surveys and should not be confused with the number of mechanical devices used in design survey procedures. Many agencies use the types of devices shown but do not necessarily use them in a rating system.

USE OF WEIGHTING FACTORS IN MAINTENANCE DECISIONS

The weights applied to distress factors by the states and agencies are largely intuitive and is apparent they can vary by significant amounts. Despite their subjective background, they are a useful summary of experience and are used in two major ways:

1. To determine a pavement rating score which is an indication of the general condition of a pavement. By using a consistent method of calculating the score the maintaining agency has a way of comparing one section of pavement with all others to determine which ones need work done on them.
2. To correlate the type or amount of distress found with the kind of maintenance that is required to return the pavement to an acceptable service level.

Some highway agencies such as California have under development a combining of these two functions and use the pavement rating score to assist in determining the type of maintenance to be done (10). Table 2.4 shows the preliminary guidelines under consideration by California. This information is only used in conjunction with other maintenance selection methods but the importance here is that such correlations are being attempted.

TABLE 2.4

PAVEMENT RATING SCORE RELATED TO MAINTENANCE

Work Type	Maximum Sum Defects	Preferred Ranges			Maximum Ride
		Sum Defects	Condition Rating	Ride	
Seal Coat	20	0-20	< 26	0-35	40
Thin Blanket	40	14-30	< 39	20-50	55
Major Maintenance	90	40-80	< 70	40-70	75
Improvement Recommendation	N/A	40-∞	> 70	70-∞	N/A

N/A - Not Applicable

The majority of highway agencies use the pavement rating score only to indicate the general condition of the pavement without attempting to tie it to any specific kind of rehabilitation. When this is the case it usually indicates that there are a variety of possible causes for a low pavement rating, each of which require a different maintenance approach.

Utah, which does not use a numerical rating scheme as yet, appears to have an excellent pavement evaluation system. Maintenance decisions are made by reviewing the results of a very detailed set of pavement measurements obtained for every section of pavement in the state. These measurements are of deflection, serviceability, skid resistance, and surface defects. Based upon this information the maintenance and rehabilitation strategy is decided upon for the coming year (11).

The proposed rating system for King County, Washington correlates the proper corrective maintenance procedure to each item of distress (12). Therefore, depending upon the extent and type of distress observed, several methods of maintenance may be considered for a pavement.

Following the example of California and King County, it would appear that a comprehensive pavement condition rating scheme would feature both an index of the general condition of the pavement and a method of determining the most effective means of rehabilitation.

STATISTICAL STUDY OF WEIGHTING FACTORS ON REHABILITATION URGENCY

In addition to determining what needs to be done in rehabilitating a pavement, another critical need in making rehabilitation decisions is to determine how soon the rehabilitation work is required. Several state agencies and pavement researchers have become interested in this subject of maintenance urgency. Ontario currently uses such a concept in their pavement rating system. Maintenance urgency is usually referred to as a prediction of the remaining life of a pavement (5, 13). The usual approach has been to assume that the remaining life is determined by the loading on the pavement and to treat pavement deterioration as a fatigue phenomenon. But the life of pavements diminishes for a variety of reasons, some associated with load but others associated with the climate, thermal and volume change properties of the paving materials, deterioration of safety characteristics and others. In order to see what kinds of factors are actually involved in an estimate of remaining life, a study of rehabilitation urgency was undertaken using visual rating data from two Texas Highway Department Districts. District 21 (Pharr, Texas) is partially in the Rio Grande river valley and near the coast of the Gulf of Mexico. The climate is warm and humid, the subgrades are largely clays and clayey sands, and there is a predominance of produce truck traffic on the rural roads. District 7 (San Angelo, Texas) is directly north of District 21 and would be considered on the boundary of the arid west Texas region. The subgrade soils are mixed, with clays and caliches being the more common. The common rural traffic is composed of farming and ranching vehicles.

All of the pavement sections in the two districts were rated using the form shown in Fig. 2.1. The columns marked waves, sags, and humps have since been replaced with a Mays Ride Meter score. The rating team was composed of two maintenance employees, one with extensive field experience and the other with office experience. After completing the visual rating of the pavement condition, the rating team was asked to put down a number between 1 and 9 which indicated the time at which it was expected that some form of

DATE ____________

LOCATION	DISTRICT NO.			
LOCATION	COUNTY NO.			
LOCATION	HIGHWAY NO.			
LOCATION	CONTROL			
LOCATION	SECTION			
LOCATION	FROM			
LOCATION	TO			
	LANE NO.			
	RATER NO.			
PAVEMENT	RUTTING	%AREA	① 1-15 ② 16-30 ③ >30	SLIGHT, MODERATE, SEVERE
PAVEMENT	RAVELING	%AREA	① 1-15 ② 16-30 ③ >30	SLIGHT, MODERATE, SEVERE
PAVEMENT	FLUSHING	%AREA	① 1-15 ② 16-30 ③ >30	SLIGHT, MODERATE, SEVERE
PAVEMENT	CORRUGATIONS	%AREA	① 1-15 ② 16-30 ③ >30	SLIGHT, MODERATE, SEVERE
PAVEMENT	WAVES, SAGS, HUMPS	%AREA	① 1 - 5 ② 6 - 25 ③ > 25	SLIGHT, MODERATE, SEVERE
PAVEMENT	ALLIGATOR CRACKING	%AREA	① 1 - 5 ② 6 - 25 ③ > 25	SLIGHT, MODERATE, SEVERE
PAVEMENT	LONGITUDINAL CRACKING	LIN. FT. PER STA.	① 0-99 ② 100-199 ③ > 200	SLIGHT, MODERATE, SEVERE
PAVEMENT	TRANSVERSE CRACKING	NO. PER STA.	① 1 - 4 ② 5 - 9 ③ >10	SLIGHT, MODERATE, SEVERE
PAVEMENT	PATCHING	%AREA	① 1 - 5 ② 6 - 15 ③ > 16	GOOD, FAIR, POOR
SHOULDER	PAVED			RIDE, CONTRAST, PAVEMENT EDGE, SHOULDER EDGE, CRACKS, RAVELING, VEGETATION
SHOULDER	UNPAVED			PAVEMENT EDGE, RUTTING, CORRUGATIONS, LOOSE ROCK
ROADSIDE				LITTER, MOWING, VEGETATION, SLOPE EROSION, ENCROACHMENTS
DRAINAGE				CULVERTS, DITCHES, OUTFALL, CHANNELS, ROADSIDE DRAINAGE
TRAFFIC SERVICES				GUARDRAILS, SIGNS, DELINEATORS, STRIPING, AUXILIARY MARKINGS, SIGNALS
	ROADWAY MAINTENANCE SCHEDULE			

FIG. 2.1 PAVEMENT RATING FORM USED IN TEXAS SURVEYS

rehabilitation would have to be done in order to prevent pavement deterioration. This rating system emphasized the <u>urgency</u> of required maintenance rather than the actual kind of rehabilitation required and is shown in Table 2.5. The urgency rating is an estimate by the rating team of the remaining life of the pavement and thus would not differentiate between a required seal coat and a pavement reconstruction job that was needed equally as urgently.

TABLE 2.5

URGENCY RATING SCALE

Numerical Rating	Description
1-2	No maintenance required two or more years
2-4	Maintenance not probable in two to five years; reevaluate within two years
4-6	Possible maintenance in one year; reevaluate in one year
6-8	Schedule maintenance within one year
8-9	Schedule maintenance immediately

Each form of distress is given two ratings: the severity which is indicated by a number ranging from 1 to 3, and the percent of the pavement area that is affected. For most forms of distress, the actual percent of the area is not recorded but is estimated to fall into one of three categories: 0-15 percent, 16-30 percent and greater than 30 percent. In the regression analysis of these data, these estimates of area were indicated by the numbers 1 to 3 corresponding to the three categories.

<u>Regression Analysis</u>. A two-step regression procedure was used to obtain equations relating the maintenance urgency rating y to the independent variables x_i: the area, the severity and the area times the severity of each kind of distress rated. The first step found exponential equations of

the form:

$$y = x_i^{\,b} - 1$$

This step was taken because it was assumed that distress and maintenance urgency would not be related linearly. The second step found the constants of a linear combination of all of the variables raised to the exponents found in step 1. The resulting model is of the form:

$$y = a_o + \sum_{i=1}^{27} a_i x_i^{\,b_i}$$

The sensitivity of this model to changes in the independent variables x_i can be determined by taking partial derivatives with respect to the x_i. Thus, if it is assumed that all of the x_i are independent (which they are not), the sensitivity equations are of the form:

$$\frac{\partial y}{\partial x_i} = a_i b_i x_i^{\,b_i - 1}$$

The larger the product $a_i b_i$, the greater will be the effect of changes in the variable x_i on causing changes in the maintenance urgency rating.

Although the product ab is an indication of the <u>importance</u> or <u>sensitivity</u> of a variable, the exponent b is an indication of how <u>suddenly</u> it can become important. The larger b is, the more rapidly the maintenance urgency rating rises once the distress reaches a critical point. If b=1, then the distress is linearly related to maintenance urgency. If b is less than 1, then as the distress grows, the urgency of maintenance becomes less sensitive to changes in the amount of distress. This case is rarely to be expected. The more usual expectation is that b will be greater than 1, since as the distress grows some kind of maintenance becomes more and more urgent. This was found to be the case in both districts, as discussed below.

<u>Regression Results - THD District 21 - 1973</u>. This was the first Texas Highway Department district which made a visual rating survey of all 353 sections of pavement in the district comprising a total of 2500 miles of road. The first step of the regression analysis of these data points was slightly different than noted above. Instead of the usual form,

$$y = x^b - 1$$

this regression analysis used

$$y = c\,x^b - 1$$

where c was allowed to be different from 1. This resulted in smaller values of the exponent b than would normally be expected but the interpretation of b and the product ab remain the same. As can be seen in Table 2.6, the most important or sensitive variables from the point of view of maintenance urgency were associated with fatigue failure of the pavement: longitudinal cracking, transverse cracking, alligator cracking, and rutting, which was apparently due to the traffic and the weaker moisture-active subgrade materials prevalent in that district. There was also strong evidence of expansive clay activity, as measured by waves, sags, and humps. Raveling proved to be a very important variable requiring immediate attention. Partly as a result of this condition survey, the District initiated an extensive seal coating program to alter the rate of pavement deterioration.

The exponent b is a measure of the variables that can cause the most rapid deterioration of pavement condition. In this analysis, these variables were rutting, alligator cracking, and longitudinal cracking, in that order. Neither flushing nor corrugations were strong enough variables to be considered in the top twenty.

<u>Regression Results - THD District 21 - 1974</u>. For the second year in a

TABLE 2.6

MAINTENANCE URGENCY RATING

District 21 - 1973
(353 Data Points)

Distress	Distress Measure	Sensitivity Rank	Exponent b	Sensitivity ab
	Severity	1	0.947	0.643
Longitudinal Cracking	Area	14	0.267	0.070
	Area x Severity	7	0.869	0.205
	Severity	8	0.704	0.179
Transverse Cracking	Area	17	0.232	0.046
	Area x Severity	4	0.623	0.313
	Severity	19	0.990	0.010
Alligator Cracking	Area	16	0.428	0.051
	Area x Severity	5	1.039	0.295
	Severity	10	0.585	0.097
Patching	Area	13	0.275	0.080
	Area x Severity	6	0.862	0.291
	Severity	3	1.050	0.431
Rutting	Area	15	0.294	0.060
	Area x Severity	12	1.061	0.088
	Severity	--	-----	-----
Waves, Sags, and Humps	Area	9	0.274	0.117
	Area x Severity	4	0.885	0.302
	Severity	--	-----	-----
Flushing	Area	--	-----	-----
	Area x Severity	--	-----	-----
	Severity	2	0.747	0.505
Raveling	Area	18	0.281	0.011
	Area x Severity	11	0.799	0.096
	Severity	--	-----	-----
Corrugations	Area	--	-----	-----
	Area x Severity	--	-----	-----

row, District 21 made a condition rating survey of all 2500 miles of roadway but this time broke it up into 722 separate sections. The regression analysis performed on these data points followed the usual form described at the beginning of this section, and the results are shown in Table 2.7. The resulting values of the exponent b are generally above 1, as expected.

It is apparent that the dominant variable has changed to flushing (or bleeding) no doubt reflecting the need for further treatment of those sections which had been seal coated the previous year.

It is apparent from the sensitivity rank of the other variables that the fatigue problem had not been overcome. Rutting, longitudinal cracking, and alligator cracking still required the most immediate attention. Corrugations, which had not been a strong variable the previous year, began to become important in 1974. None of the products of area and severity for any form of distress proved to be an important variable.

The exponents b generally fell between 2.1 and 2.5 indicating that any of the variables have the capability of causing rapid deterioration of the pavement condition.

Regression Results - THD District 7 - 1974. In 1974, District 7 undertook a complete condition rating of all 2700 miles of roadway in the district composed of 369 separate sections. The regression analysis followed the usual form. Results from the analysis are shown in Table 2.8.

Two of the most sensitive forms of distress were longitudinal cracking and waves, sags, and humps. The conjunction of these two as important variables indicates the presence of an active subgrade material and volume change due to moisture instability (shrinking or swelling) beneath the pavement. The surface condition of the pavement as measured by patching,

TABLE 2.7

MAINTENANCE URGENCY RATING

District 21 - 1974
(722 Data Points)

Distress	Distress Measure	Sensitivity Rank	Exponent b	Sensitivity ab
Longitudinal Cracking	Severity	8	2.421	0.453
	Area	3	0.676	1.739
	Area x Severity	--	7.988	-----
Transverse Cracking	Severity	7	2.382	0.488
	Area	11	2.271	0.223
	Area x Severity	--	6.506	-----
Alligator Cracking	Severity	5	2.396	1.459
	Area	--	-----	-----
	Area x Severity	--	-----	-----
Patching	Severity	14	2.103	0.025
	Area	15	2.437	0.010
	Area x Severity	--	-----	-----
Rutting	Severity	4	2.497	1.488
	Area	2	2.530	1.854
	Area x Severity	--	8.063	-----
Waves, Sags, and Humps	Severity	9	2.482	0.325
	Area	10	2.532	0.299
	Area x Severity	--	8.027	-----
Flushing	Severity	1	2.525	2.507
	Area	--	-----	-----
	Area x Severity	--	-----	-----
Raveling	Severity	12	2.455	0.221
	Area	13	2.467	0.032
	Area x Severity	--	7.503	-----
Corrugations	Severity	6	2.528	0.554
	Area	--	-----	-----
	Area x Severity	--	-----	-----

TABLE 2.8

MAINTENANCE URGENCY RATING

District 7 - 1974
(369 Data Points)

Distress	Distress Measure	Sensitivity Rank	Exponent b	Sensitivity ab
Longitudinal Cracking	Severity	1	1.697	0.796
	Area	2	1.653	0.633
	Area x Severity	17	3.491	0.007
Transverse Cracking	Severity	13	1.709	0.073
	Area	9	1.654	0.210
	Area x Severity	18	3.521	0.003
Alligator Cracking	Severity	8	1.884	0.271
	Area	--	-----	-----
	Area x Severity	--	-----	-----
Patching	Severity	3	2.037	0.530
	Area	15	1.869	0.022
	Area x Severity	--	-----	-----
Rutting	Severity	11	1.966	0.122
	Area	10	1.904	0.164
	Area x Severity	--	4.829	-----
Waves, Sags, and Humps	Severity	4	1.507	0.496
	Area	6	1.358	0.410
	Area x Severity	16	2.848	0.011
Flushing	Severity	7	1.889	0.306
	Area	12	1.618	0.102
	Area x Severity	--	3.896	-----
Raveling	Severity	5	1.948	0.438
	Area	--	-----	-----
	Area x Severity	--	-----	-----
Corrugations	Severity	14	2.051	0.029
	Area	--	-----	-----
	Area x Severity	--	-----	-----

raveling, and flushing is also a strong indicator of the urgent need for maintenance work. As in the previous analysis of District 21 data, none of the products of area and sensitivity of any form of distress was a significant variable. This may be an indication that the two measures, i.e., area and severity, may have virtually no interaction, and consequently may be treated independently in a model of pavement condition deterioration.

The exponent b had a fairly narrow range between 1.6 and 2.1, again indicating that any of the forms of distress can cause rapid deterioration of the pavement condition. The size of b is lower in this district than in District 21. There is not enough experience with this kind of analysis to tell whether this difference in b reflects a difference in climate or simply a subjective difference in the rating teams.

THE MOST SENSITIVE PAVEMENT VARIABLES FOR REHABILITATION URGENCY OF A PAVEMENT

This study of the sensitivity of rehabilitation urgency to various pavement condition variables was conducted in Texas where the soils, climate, and traffic may not be typical for all other locations within the United States. The list of most sensitive variables is expected to change with geographical location. Nevertheless, the list given in Table 2.9 indicates a certain amount of consistency.

Both in 1973 and 1974 in District 21, rutting and all forms of cracking were dominant variables. In District 7, again cracking was dominant but it was mainly longitudinal cracking in conjunction with waves, sags, and humps. The condition and appearance of the pavement surface as measured by raveling, patching, and flushing were strong variables in both districts indicating a concern for the safety and aesthetic interests of the traveling public.

TABLE 2.9

MOST SENSITIVE PAVEMENT CONDITION VARIABLES
(TEXAS STUDY)

Sensitivity Rank Variable	District 21 1973	District 21 1974	District 7 1974
1	Long. Cr. Severity	Flushing Severity	Long. Cr. Severity
2	Raveling Severity	Rutting Area	Long. Cr. Area
3	Rutting Severity	Long. Cr. Area	Patching Severity
4	Transv. Cr.(Area x Sev.)	Rutting Severity	Waves Severity
5	All. Cr. (Area x Sev.)	All. Crack. Severity	Raveling Severity
6	Patching (Area x Sev.)	Corrugation Severity	Waves Area
7	Long. Cr. (Area x Sev.)	Transv. Cr. Severity	Flushing Severity

There were differences between the two districts also, and these differences indicate the importance of the produce truck traffic in the Rio Grande Valley. The conjunction of alligator cracking and rutting in District 21 shows the dominant influence of a load-associated deterioration of pavement strength in the Rio Grande Valley. Similarly, the conjunction of longitudinal cracking and waves, sags, and humps in District 7 shows that the San Angelo district has an active expansive clay subgrade problem.

The fact that the maintenance urgency rating shows up these variables very strongly demonstrates the fact that maintenance urgency will not only change in response to maintenance activity (as in the previous two examples) but it will also change with the important environmental characteristics -- including the type of subgrade which underlies the pavement in a given district.

CONCLUSIONS

On the basis of this study of pavement condition rating systems, it is apparent that the most important distress variables related to the structural condition of a flexible pavement are rutting and the various forms of cracking: longitudinal, transverse, and alligator. For rigid pavements, the important variables are cracking, spalling, and faulting. These are not all of the determining factors in establishing the remaining life of a pavement but they are always indicators of structural distress that determines the remaining structural life of a pavement. The other factors that determine the remaining life of a pavement are the riding quality (related to roughness), and the surface characteristics (related to skid resistance and the surface distress modes).

In the United States, rutting is measured by relatively simple devices such as a straight edge and measuring rule or dial gage; whereas in some foreign countries, a traveling device that records several profiles abreast at various locations across the wheel path is employed. It appears that the straight edge and rule device provides sufficient information for the rapid or decision survey.

Very little equipment has been developed to count cracks. Aerial photography or moving picture photography is used to record the cracked condition of a pavement but some laborious form of visual data takeoff is required to determine the actual number of cracks. In some cases, the length of visible cracking within a specified area is measured to determine the extent of cracking. Cracking appears to be the most important single distress variable and despite this fact no means other than visual observation is available to count them rapidly and automatically.

One reason for this is that no measurement system other than visual observation is reliable enough to determine the cracked condition of a pavement. Another reason is that any crack counting instrument that travels down the road would be able to record only the transverse cracks it crosses. Alligator cracking could be inferred from the close spacing of the transverse cracks but the length of longitudinal cracking would be difficult to infer from the output of a traveling crack counter.

The major benefit of having such equipment would be that it would collect information on crack spacing rapidly and economically, would not require an extensive effort at data reduction, and would not flood the highway agency with excessive amounts of superfluous data. In addition, it appears that information on crack spacing will be an essential element in future design methods for overlays to prevent reflection cracking (5). The thickness of the overlay will be fairly heavily dependent on crack spacing and this makes it important to have a fairly reliable determination of crack spacing. Furthermore, if crack spacing can be measured reliably by an instrument, then a visual observation of the extent and severity of longitudinal cracking will give all of the structural distress information required for making decisions on which pavement sections will need some form of rehabilitation.

Pavement condition rating systems appear to be excellent diagnostic tools. Their reliability and consistency can probably be improved by measuring some of the more critical distress variables in a less subjective way, possibly by the use of a measuring instrument. The required rehabilitation is expected to be correlated consistently with a given combination of types and severities of distress. Since the pavement rating score is a weighted average of <u>all kinds</u> of distress, it is <u>not</u> expected to correlate consistently with required rehabilitation. Instead, the pavement rating score should most consistently reflect the rehabilitation urgency, a measure of the remaining life of a pavement. The pavement rating score -- or the rehabilitation urgency -- will respond to the influence of a variety of factors some of which may be important one year and not important the next.

It is apparent that the results of a pavement condition rating may be used in two ways: for diagnosis to determine what needs to be done, and for prognosis, to predict how soon it must be done.

CHAPTER III

HIGHWAY TECHNOLOGY

Highway technology has produced a significant number of non-destructive pavement evaluation techniques. Some of these are production models that are in daily use by various highway agencies. Others are still in the development stage and while their principles of operation are known and the data they produce can be used in several ways, few of them produce data which can be analyzed to produce material properties of the pavement layers. Appendix B presents a detailed description of this array of equipment, their principles of operation, their capabilities of producing analyzable data, and their advantages and disadvantages for applications in pavement evaluation. This chapter has a different objective and that is to consider the relative merits of these items of equipment in meeting the criteria for decision surveys and design surveys established in Chapter I. Other promising pieces of equipment which might be developed will also be considered in this assessment.

The criteria established in Chapter I are as follows:

<u>Decision (Rapid) Survey</u>

Speed of operation
Repeatability of the measurement
Consistency of the measurement between roadway sections
Capability of correlation with Benkelman beam data

<u>Design (Detailed) Survey</u>

Repeatability of the measurements
Analyzability of measurements to determine material properties
Applicability of the results to
  a. present design procedures
  b. future design procedures including some that are being developed at present
  c. design procedures which include the effects of the environment on the material properties and the performance of the pavement.

In decision surveys, the importance of the speed of operation is not on speed per se, but on getting sufficient data in a short time to determine the current condition of all of the elements of a roadway network. These surveys are made each year and in some cases twice a year for the purpose of deciding what kind of maintenance and rehabilitation strategy to apply to the roadway network; which sections of roadway need work more than any others and what kind of work needs to be done. The decision is made based on the data available. If the data are not reliable or if they are not consistent from one section to the next, then the likelihood of making poor decisions will increase. Consequently, a decision survey is interested in gathering <u>reliable</u>, <u>consistent</u> data <u>rapidly</u> and is concerned with the speed of operation of the equipment only insofar as it can produce the required data when it is needed.

In the design surveys, the applicability of the measured results to design procedures is the most important characteristic. The elastic and viscoelastic moduli of each pavement layer and the cracked condition of the surface course are, in general, of most importance to any pavement analysis method. Even empirical pavement design procedures can use these material properties if their important measure(s) of pavement performance (e.g. deflection, surface curvature index, base curvature index, or other basin properties) are computed from a layered analysis scheme. As a general comment, it appears that most of the non-destructive pavement evaluation schemes have been using loads, load configurations, frequencies, and so on, that most reliably indicate the properties of the <u>subgrade</u>. This is unfortunate, to some extent, because the single layer that best indicates the condition of the pavement and the one that most can be done with in design, construction, and rehabilitation is the surface course. Consequently, a major emphasis in the considerations of this chapter will be the effectiveness of the various evaluation methods for determining the properties and cracked condition of the surface course.

The kinds of data that must be collected in the two kinds of survey are different, a reflection of their different purposes. Decision surveys are concerned with distress and design surveys are concerned with material properties, crack spacing and severity, and response of the pavement structure to imposed loads or environmentally-induced stresses. The following is a list of what data each of the surveys may assemble.

Decision Survey

- Deflections
- Stiffness
- Cracking
- Rutting
- Roughness
- Skid resistance

Design Survey

- Deflections
- Cracking
- Layer moduli

Both surveys are interested in deflections mainly because of the need to link what is measured with performance data which, in turn, has been related to Benkelman beam measurements. The interest in cracking in each survey is different, however. The decision survey is interested in the amount of cracking and how severe it is, whereas the design survey is interested in more detail. For design purposes, the crack spacing, size, and location within the pavement structure is important.

The order in which decision survey equipment will be discussed was listed above: deflections, stiffness, cracking, rutting, and roughness. Skid resistance is an important property of the pavement surface but does not contribute to the structural condition of the pavement and will not be considered here. Equipment to measure deflections and stiffness will be considered together.

Decision Surveys of Pavement Stiffness

There are two methods of conducting a decision survey of pavement stiffness and each has its own merits:

1. Mass Inventory. Make many measurements along the pavement so as to discover the location of the weak points that are most in need of repair. Presumably, the pavement section with the greatest density of weak points in the roadway network would receive maintenance attention earlier than one with a lower density.
2. Statistical Sampling Study. Make sufficient measurements to determine a reliable statistical distribution of pavement stiffness. Presumably, the pavement sections with the lowest average and greatest spread (as measured by the standard deviation) would receive the earliest rehabilitation efforts.

In either case, the objective is to establish rehabilitation priorities among several candidate sections in a roadway network. In the mass inventory, sufficient data are gathered to pin point places for spot patching. This is the kind of rapid survey which would be conducted by the California Traveling Deflectometer or the Lacroix Deflectograph which have the capability of making 1000 to 4000 measurements a day. The data that are produced are a collection of stiffness numbers that may or may not be well correlated with Benkelman beam data and may not mean the same thing on one pavement section as it does on another. What it does give is an estimate of how stiff one spot of pavement is relative to adjacent spots of pavement along the same length of road.

In the statistical sampling study, the emphasis shifts toward collecting data that can be analyzed to determine elastic moduli, coefficients of subgrade reaction, or other similar material properties of the pavement while obtaining a reasonably reliable statistical distribution of pavement stiffness numbers. These numbers may or may not represent material properties. In some cases, various measurements taken within a Dynaflect basin, such as SCI (surface curvature index), BCI (base curvature index) and DMD (dynaflect maximum deflection) are used as a measure of pavement stiffness (14). In other cases, elastic moduli may be calculated from the measurements of surface deflections (15). Because this approach is slower and makes fewer measurements per day, it loses the detail that can be achieved with the methods that can make a mass inventory of pavement stiffness. Nevertheless, the statistical sampling approach still achieves the major objective of the survey which is to collect data from which rehabilitation decisions can be made. Furthermore, it has the advantage that the data can be analyzed to determine the distribution of material properties along the length of a pavement.

In net balance, the adaptability of the statistical approach using slower equipment with analyzable data is expected to demonstrate greater and more cost effective long-range benefits.

A study of the statistical approach was conducted using Dynaflect data which was measured every one-half mile over a 100-mile length of rigid pavement on I.H. 45 between Houston and Dallas. A series of two computer programs were written to analyze the data. The first is the analysis program which used Westergaard's equations for the deflections of a point load on a rigid pavement resting on a liquid subgrade (16) to determine the elastic modulus, E, of the concrete and the subgrade modulus, k, of the subgrade.

The equation for surface deflections, w, is of the form:

$$w = \frac{P}{kl^2} \; f\left(\frac{x}{l}\right)$$

where

P = the size of the point load

k = the subgrade modulus

x = the distance away from the point load

l = the radius of relative stiffness.

$$= \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4}$$

E,μ = the elastic modulus and Poisson's ratio of the concrete

f = a decreasing function of x/l.

The technique used chooses E and k by trial and error to minimize the sum of the squared errors between predicted and observed deflections. The second program determines the statistical properties of the calculated E and k values along the road. This program then drops out data in a specified pattern so that 90 percent, 80 percent, 70 percent and smaller size samples can be used to calculate the same statistical properties, which include the mean, standard deviation, skewness, and kurtosis of the distribution. By finding the smallest size of sample that produces about the same statistical properties, one locates the minimum sampling rate for a pavement survey.

Fig. 3.1 shows a typical distribution of the elastic modulus of the

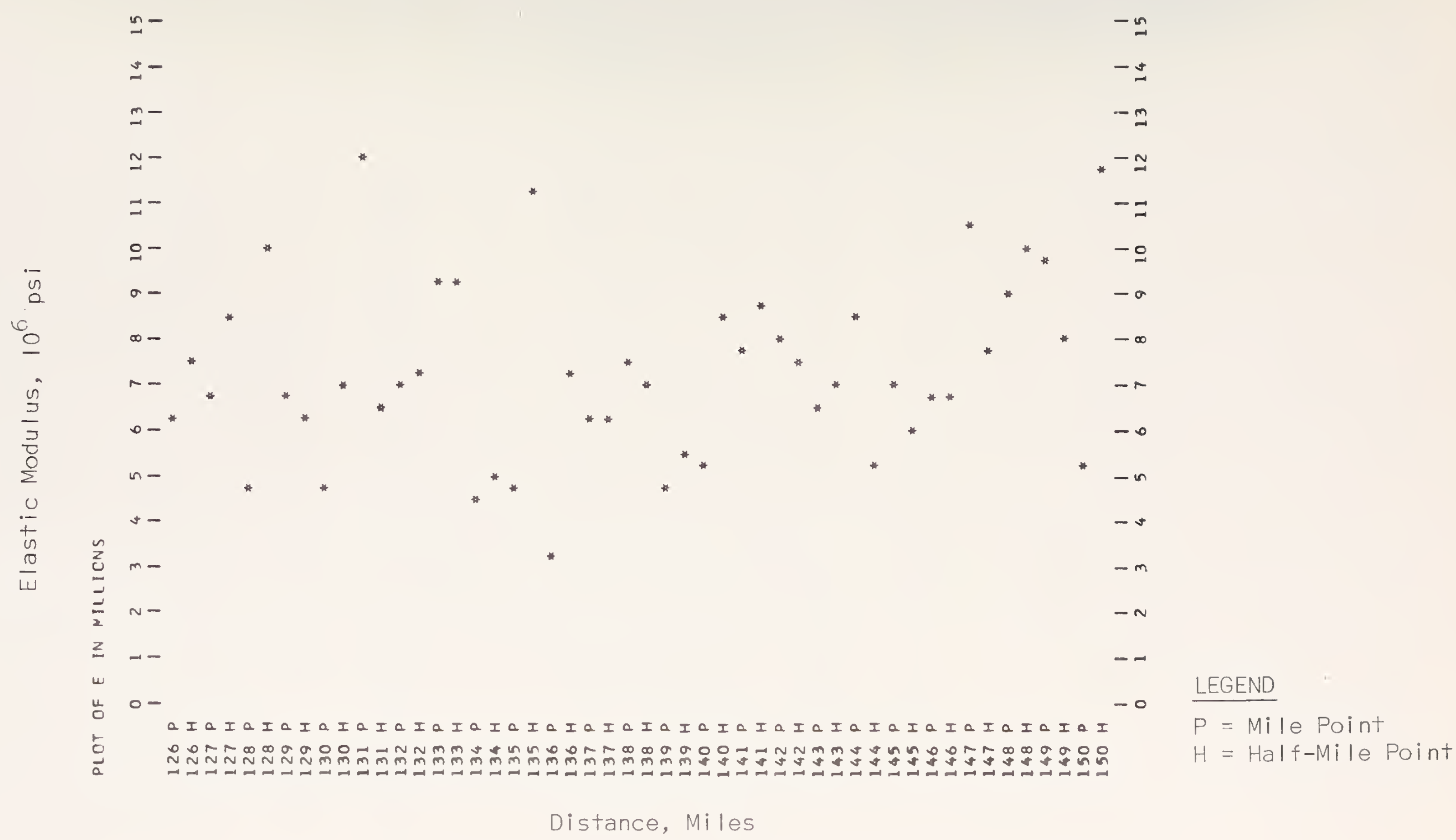


FIG. 3.1 VARIATION OF CONCRETE ELASTIC MODULUS ALONG 25 MILES OF I.H. 45 BETWEEN HOUSTON AND DALLAS

concrete pavement over a 25-mile length of pavement. The values of E range between about 3 and 12 x 10^6 psi. The higher values are undoubtedly in error probably due to an underestimate of the thickness of the pavement or to the presence of a stiff sub-base material which has the same effect on the analysis as underestimating the thickness of the concrete. This is confirmed, to some extent, by Fig. 3.2 which shows the values of the subgrade modulus over the same length of road. The larger values of E are in roughly the same location as the larger values of k indicating the possible presence of a three-layer pavement which is insufficiently well modeled by the two layer Westergaard equation.

The statistical program then sampled the calculated data and produced the statistical measures of elastic modulus shown in Table 3.1. The total number of samples considered was 180. Skewness measures the distribution of the data around the mean and kurtosis measures how peaked the distribution is. A value of zero in each case is a property of the normal distribution.

TABLE 3.1

STATISTICAL PROPERTIES OF ELASTIC MODULUS DISTRIBUTION*

	Size of Sample (in percent)				
	100	80	50	30	10
Mean	6.84	6.76	6.76	6.74	6.70
Standard Deviation	1.61	1.63	1.65	1.72	1.75
Skewness	-0.105	-0.106	-0.207	-0.133	-0.193
Kurtosis	-0.54	-0.58	-0.48	-0.32	-0.67

*All figures are in units of 10^6 psi.

FIG. 3.2 VARIATION OF SUBGRADE MODULUS ALONG 25 MILES OF I.H. 45 BETWEEN HOUSTON AND DALLAS

The 50 percent sample, representing a measurement every mile, gives values that are nearly identical with those of the 100 percent sample. Even the 10 percent sample, computed from only 18 measurements, gives acceptably close values of the mean and standard deviation. This 10 percent sample represents a measurement made every five miles. A similar determination was made for the subgrade modulus distribution. While this is not suggested as standard practice, it does show that relatively infrequent measurements can produce acceptable statistical measures of pavement properties. Furthermore, it indicates that a study such as this can sometimes greatly reduce the amount of data required for making decisions on rehabilitation and at the same time produce data that are sufficiently accurate for the design of overlays and other forms of pavement rehabilitation.

These considerations demonstrate that the speed of operation of deflection or stiffness measuring devices, or in fact, any kind of device, is relatively unimportant as long as the equipment can be used effectively as part of a statistical sampling survey.

Impulse and Impedance Methods

Among the methods of determining pavement stiffness are the impulse testing techniques developed at the Cornell Aeronautical Laboratory (CAL), the Washington State University (WSU), the Phoenix Falling Weight Deflectometer (PFWD), as well as the vibration testing impedance technique developed in South Africa at the National Institute for Road Research of the South African Council for Scientific and Industrial Research (NIRR). All of these devices are capable of making measurements which can be analyzed provided that both input force and output response are measured as a function of time. The WSU device measures vertical accelerations as the pavement output response with time while the NIRR device measures vertical velocities as the output response and the CAL and PFWD devices measure displacements with time. Because of the way they operate, these devices are well-suited to a statistical sampling survey. The vehicle-mounted WSU device is even capable of collecting data on a mass inventory basis. The only questions that arise about these devices are:

1. Are the measurements consistent from one pavement section to the next?
2. Can the data be analyzed to determine layer material properties for design purposes?

Both of these questions may be answered by considering how the data can be analyzed. Szendrei and Freeme (9) define the pavement impedance function, $Z(\omega)$, as the ratio of the Fourier transforms of the input force and the output velocity response:

$$Z(\omega) = \frac{\int_{t=-\infty}^{t=\infty} f(t)\, \exp(-j\omega t)dt}{\int_{t=-\infty}^{t=\infty} v(t)\, \exp(-j\omega t)dt}$$

where

$f(t)$ - the input force as a function of time

$v(t)$ - the output velocity response as a function of time.

ω = the frequency in radians/sec.

The function $Z(\omega)$ can be determined by performing discrete Fourier transforms on the force and velocity separately, each resulting in a complex number at each frequency used in the transform. Complex division of the force transform by the velocity transform at each frequency results in the complex impedance $Z(\omega)$ at that frequency. If $F(\omega)$ is the Fourier transform of the forcing function f(t) at a point, then the displacement at that point is

$$x(t) = \int_{w=-\infty}^{w=\infty} \frac{F(\omega)}{j\omega\ Z(\omega)} \exp(j\omega t) dz\omega$$

and the acceleration is

$$a(t) = \int_{w=-\infty}^{w=\infty} \frac{j\omega(F(\omega)}{Z(\omega)} \exp(j\omega t) d\omega$$

Using the derived impedance function $Z(\omega)$ and the Fourier transform of the force function $F(\omega)$, and the exponential rate of attenuation of $Z(\omega)$ with distance from the loading point, $\alpha(\omega)$, and its phase retardation with distance, $\beta(\omega)$, the deflection response of a pavement surface to a passing load can be calculated as demonstrated by Szendrei and Freeme (9). This same impedance function could be derived from the WSU measurements using the following relation at every point where acceleration, a(t), is measured.

$$\frac{Z(\omega)}{j\omega} = \frac{\int_{t=-\infty}^{t=\infty} f(t) \exp(-j\omega t) dt}{\int_{t=-\infty}^{t=\infty} a(t) \exp(-j\omega t) dt}$$

A succession of these determinations with distance away from the point of load application will allow the exponential attenuation rate, $\alpha(\omega)$, and the phase retardation, $\beta(\omega)$, to be calculated. Thus, approximately the same analysis techniques developed in South Africa could be used to analyze data from the WSU device and this would permit a calculation of the deflection response of a pavement to any selected moving load.

Both the CAL and the PFWD measure output displacement response at only one point immediately beneath the load so that it would be impossible to use their data to determine $\alpha(\omega)$ or $\beta(\omega)$. Nevertheless, these data can be used to obtain the impedance function provided that both f(t) and x(t) are measured and used as follows:

$$j\omega Z(\omega) = \frac{\int_{t=-\infty}^{t=\infty} f(t)\ \exp(-j\omega t)dt}{\int_{t=-\infty}^{t=\infty} x(t)\ \exp(-j\omega t)dt}$$

If moving load predictions were required, then one other piece of equipment would be needed. Either another displacement transducer placed away from the load or, following the NIRR method, a separate set of measurements using wave propagation apparatus will give the attenuation exponent $\alpha(\omega)$ and the phase retardation with distance $\beta(\omega)$.

In principle, all of these devices may be considered together in determining their potential usefulness to pavement evaluation.

Despite the obvious success of the method in determining the overall stiffness of a pavement and in predicting pavement deflection response to moving loads, its major drawback to the present time has been its inability to determine the material properties of the different layers in a pavement system. Once this kind of analysis is developed, the relative accuracy of the impulse and impedance methods may be compared with the dynamic deflection methods now in use and the better one selected for future use. In the meantime, these methods can provide an indication of pavement stiffness which may not be consistent from one section to the next and may not be able to provide information that is immediately useful for design. It is possible that some empirical correlation may be achieved with Benkelman Beam data but the consistency of this correlation from one pavement section to the next should be investigated carefully. Since the CAL, WSU, and PFWD devices all measure response at several places away from the load, they may be useful in detecting cracks. A crack will sharply attenuate the response of the pavement to the applied load. Because of this, the WSU travelling device offers promise of making statistical sampling surveys of cracks and crack spacing for the purposes of a decision survey.

Cracking Surveys

Decision surveys are mainly interested in the current condition of the pavement as it will affect future maintenance or rehabilitation work. As shown in Chapter 2 and Appendix A, the cracked condition of a pavement accounts for a major portion of the pavement rating score and, as such, is the prime indicator of the need for work to be done on the section. The length of longitudinal cracking and the area of alligator cracking can be estimated nearly as well. Despite the importance of such cracking, this study found that there were no instruments within highway technology that were being used to count cracks. However, some types of existing equipment could be used in unusual ways to count cracks. One of these was actually tried out and the other two are still in the conceptual stage.

The one that was tried out and did prove to be very useful was the GM Profilometer. The study of crack counting with the profilometer is presented in detail below. The other two methods used: (a) the existing Dynaflect equipment with an impulsive loading device and (b) a new configuration of the rolling delamination detector (17). Both of these concepts, their likelihood of successful use, and the best method of using them is discussed below.

Crack Counting with the GM Profilometer

The GM Profilometer is capable of making very accurate detailed measurements of pavement profile in the right and left wheel paths. Usually the analog measurements made by the profilometer are converted to digital data with a profile elevation given every 2.027 inches along the roadway. The entire process is described in detail in Refs. (18, 19). A computer plot of typical profilometer data is shown in Figure 3.3 where each asterisk represents an elevation change of about 0.015 inches. The numbers marked at the bottom of the figure are the distances in feet from the beginning of the profile which was measured along a test-section of badly cracked flexible pavement on I.H. 20 in Texas Highway Department District 6. The cracking along this length of pavement is apparently caused by thermal shrinkage of the base course.

The large dip which is centered on distance 17.22 ft. is a crack which is about 0.28 inches deep. The really significant feature of this crack is the depression on each side of it. As expected from analysis, (Ref. 20), a shrinkage crack in the base course will draw down the pavement on each side of it for a considerable distance which in this case is about 1.5 ft. The characteristic V-shape of a crack makes it a visually distinctive feature in a profile of a flexible pavement. A crack in a rigid pavement where the surface course is a brittle material will be much more abrupt. In either case, the crack may become accentuated with time as fines are pumped out of the base course. Distortion around the crack will always point toward the most active layer - the layer that has caused the crack.

The observation of the V-shape around a crack led to the development of a special profile filter which distinguishes a V-shape and stores in computer memory the location of the center of the crack. The profile

* - 1 Asterisk = .015 inches

FIG. 3.3 TYPICAL GM PROFILOMETER DATA SHOWING A CRACKED PAVEMENT PROFILE

filter first smooths the profile by averaging the five points centered around a given point and then manufactures an even smoother profile by averaging 30 points around the given point. A crack is defined by a difference in elevation between the five-point averaged and the 30-point averaged profiles. The 30-point averaged profiles provides a relatively smooth datum with which to compare the 5-point profile while following the general slope of the pavement fairly faithfully. The five-point averaging was done to eliminate extraneous "hash" from the profile.

After observing a number of cracks along the profile, it was determined that the following criteria adequately describe a crack:

1. The difference in elevation between the five-point and 30-point profile must be greater than 0.06 inches (4 asterisks).
2. The surface profile must go down into the depression and come back up within a total distance of 4 feet.

By trial and error, it was found that using criterion 1 alone gave the same crack information as obtained when using the two criteria together. Consequently, criteria 2 was eliminated for this study. Obviously, the same filter can be used for cracks that curl upward on each side of the crack. This kind of cracking is typical of areas where the surface course is the most thermally active layer in the pavement.

The crack counting filter found that there are two cracks in the space displayed in Figure 3.3

1. A crack of severity 4 (0.06 inches) at 11.48 feet.
2. A crack of severity 19 (0.28 inches) at 17.22 feet.

A frequency distribution of the cracks found within an 800-foot distance is shown in Figure 3.4. A total of 78 cracks were found with this filter, which gives an average crack spacing of just over 10 feet.

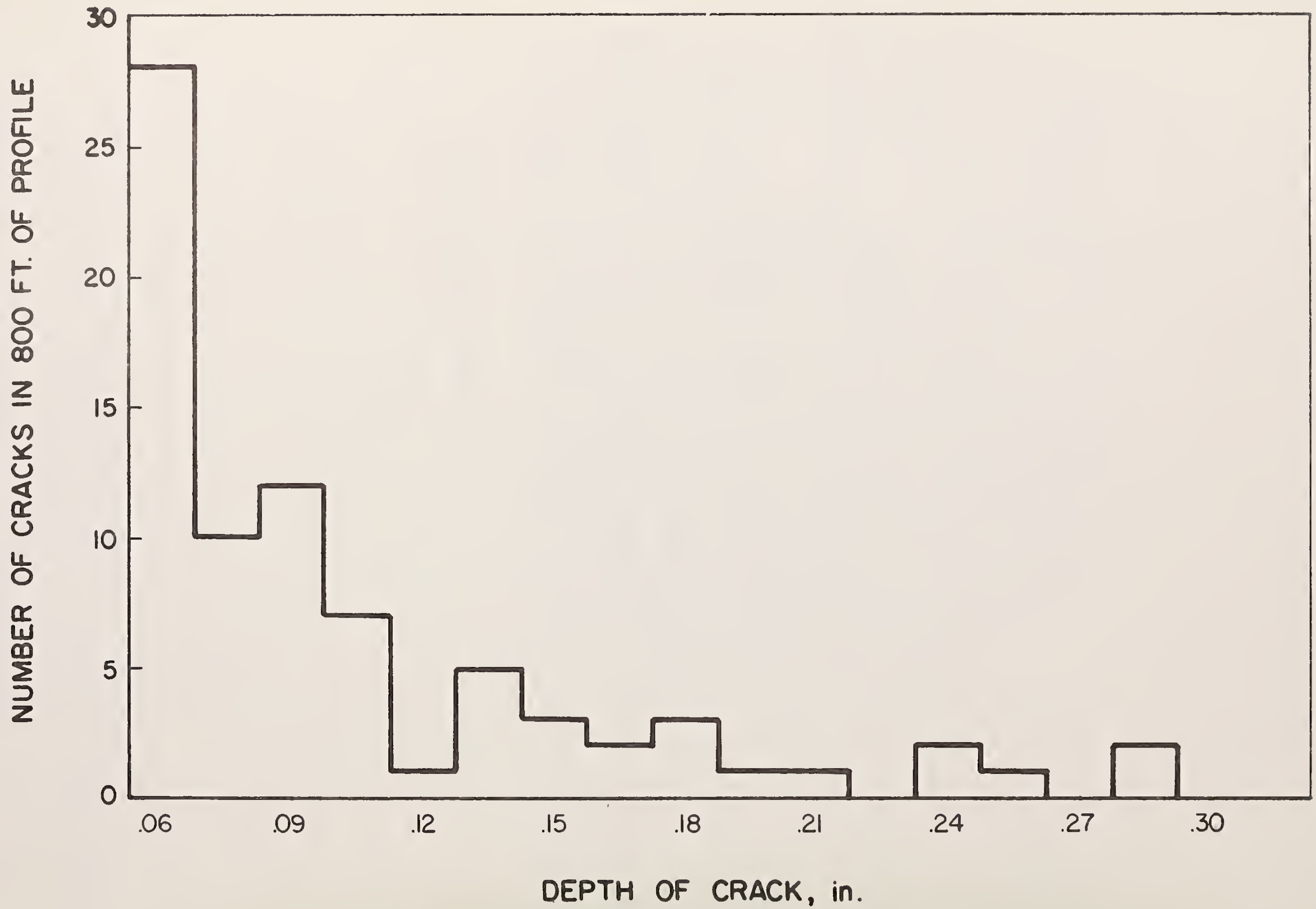


FIG. 3.4 FREQUENCY DISTRIBUTION OF CRACK DEPTHS ALONG I.H. 20, BETWEEN MIDLAND, TEXAS AND ODESSA, TEXAS

A field survey of this same section of pavement indicated that the visible cracks occur on the average of 12 feet, a reasonably close match.

Although the difference between a 10 and a 12-foot crack spacing may be only a statistical error, it does suggest that the crack-counting filter found some 11 out of 78 surface profile features that resembled cracks, but may not have been.

There are two possible interpretations of this finding:

1. The crack counting filter is in error and should use a greater difference in elevation as a crack criterion. An elevation difference of 0.08 inches would give an average crack spacing of about 16 feet.
2. The crack counting filter has found some cracks which are as yet invisible.

It is impossible at this stage to say which of these interpretations is correct, a determination that will require further field investigation. Analytical results such as those in Ref. (20) show clearly the mechanism of pavement depressions forming above where cracks in the base course have not yet broken through the surface. Whether such a depression will always indicate the presence of an invisible crack is another question that remains to be determined.

It can certainly be concluded that the crack counting filter is a convenient, automatic, and fast method of determining cracks from GM profilometer data. It may have a hidden potential for detecting invisible cracks.

Crack Detection with the Dynaflect

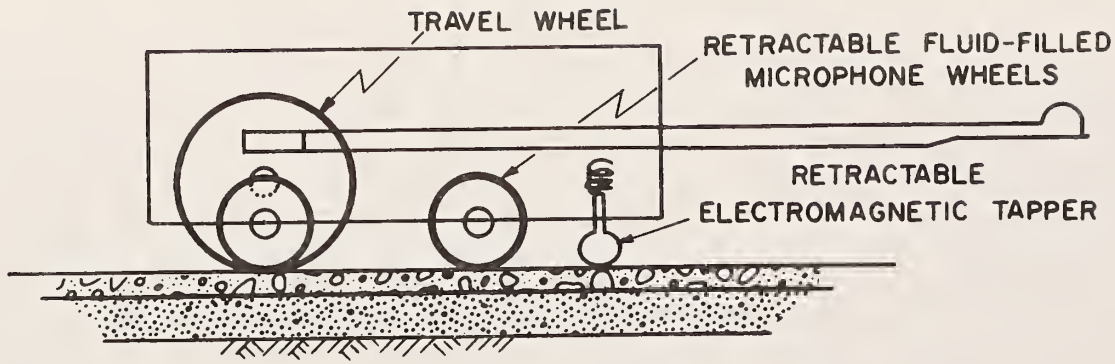
The investigation of the Washington State University device which employs an impulsive load on the pavement and the measurement of accelerations at several points away from the load immediately suggested an important use for the Dynaflect. By mounting an impulsive loading device on the Dynaflect and attaching an accelerometer to it to measure the applied force with time, then the Dynaflect geophones could measure the pavement velocity response with time. Fourier transforms could then be used to derive an impedance function $Z(\omega)$, an attenuation exponent $\alpha(\omega)$, and a phase retardation $\beta(\omega)$ as described earlier in this chapter.

However if a crack existed between any two geophones, the rate of attenuation would be increased sharply for all frequencies. Thus, whether a crack is visible or hidden, its presence could be detected by the attenuation between geophones. The five geophones are spaced one foot apart and consequently, in a matter of seconds, a four-foot long section of pavement could be sampled to determine the integrity (absence of cracks) of the pavement. However, since there is some doubt whether this four-foot length would be a statistically reliable sample of the pavement, it is likely that several such samples should be taken at a given location over a distance that spans the expected crack spacing plus at least two standard deviations. For concrete pavements and pavements on stabilized bases courses, this may be between 10 and 20 feet. For continuously reinforced concrete pavements or pavements subject to severe thermal cracking this may be between 3 and 10 feet. Regardless of the type of pavement, it is apparent that at least two and possibly five or six consecutive four-foot lengths would have to be sampled at each location to make certain that an actual cracked condition is detected. This procedure would have to be followed at least at every location where Dynaflect readings are made in order to get a reliable a determination of the pavement integrity.

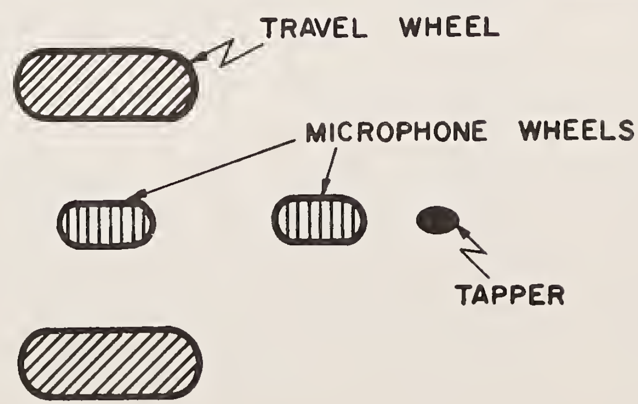
In spite of the attractiveness of making double use of the Dynaflect geophones to determine deflection basins and to detect cracks, the extra time that would be required to make several set ups for crack detection at each location may reduce the production rate of the Dynaflect to an unacceptable level. Consequently, some other scheme available from current highway technology was sought to measure pavement integrity more rapidly.

Crack Detection with a "Mobile Acoustic Crack Detector"

A modification of an existing device to measure delamination in bridge decks with acoustic signals (17) could be used to detect cracks. The existing delamination detector "taps" on the bridge deck with an electromagnetic "tapper" and picks up the signal transmitted through the bridge deck with microphones mounted inside rolling rubber wheels filled with fluid. A prototype of this device has been built, field tested, documented and was found to be a reliable method of detecting the horizontal delamination cracks in bridge decks at speeds over 10 mph. Speed appears to produce no distortion in the signal so the only reason for maintaining a lower speed is to eliminate wheel bounce. The same concept can be used to detect transverse cracks in a pavement. Two fluid-filled wheels placed one behind the other can be arranged in a line behind an electromagnetic tapper. The acoustic signal from the tapper would be picked up first in Wheel No. 1 and then in Wheel No. 2 as shown in Fig. 3.5. Attenuation of the signal can be measured directly and a sharp increase of signal



A. MOBILE ACOUSTIC CRACK DETECTOR IN OPERATION



B. WHEEL CONFIGURATION

FIG. 3.5 CONCEPTUAL DRAWINGS OF A MOBILE ACOUSTIC CRACK DETECTOR attenuation indicates the presence of a crack between the two wheels. A certain amount of experimentation would have to be done to determine the spacing of the wheels and the strength of the tapper to get an optimum arrangement for detecting cracks. In addition, some tests would have to be made to determine the maximum speed at which the device could operate and still maintain acoustic contact with the pavement. This device can be trailer mounted and towed behind the Dynaflect. While a basin is being measured, the travel wheels can be retracted so that the weight of the trailer rests on the fluid-filled microphone wheels. After the basin has been measured, the mobile acoustic crack detector can be pulled forward at maximum speed for about 100-200 feet, count the cracks, and measure their spacing with an odometer. Then, while still traveling, the travel wheels can be lowered and the towing vehicle speed can be increased to normal highway traveling speed. This should make the measurement of pavement stiffness (deflection basin) and integrity (crack counting) almost as rapid an operation as can be done at present measuring stiffness alone.

The successful use of the fluid-filled microphone wheels in the Delamination Detector suggests that their use in a Mobile Acoustic Crack Detector may be equally successful as a device for use in decision surveys. The speed with which they can be operated is a significant factor in their favor as is their ability to count both visible and hidden cracks.

Other Types of Distress in a Decision Survey

The other critical types of distress to be measured in a decision survey include rutting and roughness. A statistical sampling survey of rut depth using a simple curvature meter (see Appendix B) or other such device that can be operated manually should be sufficient to get reliable data for rut depth. Roughness is indicated by various measurement devices mounted in a vehicle. These include the PCA meter, the Mays Meter, and the truck-mounted slope-variance indicator used in Colorado. This type of equipment has proven to be rapid and reliable and has been implemented in various states. It is concluded that no new developments are needed for decision survey equipment to measure rutting or roughness. Skid resistance equipment evaluates the surface friction characteristics of a pavement and is beyond the scope of this study.

DESIGN SURVEY EQUIPMENT

As noted in Chapter I, a design survey is undertaken to collect detailed data on the material properties of the layers and the geometry of distress patterns in order to make a structural design of the rehabilitation work that needs to be done. Several design procedures are empirical and are based upon deflections while others are based upon elastic or viscoelastic theory. Still other design methods use the length between cracks (or joints) in an existing pavement as one factor in the design of overlays to prevent reflection cracking.

The items of equipment that are available for these purposes are described in Appendix B. The major findings contained in the appendix will be reviewed here.

Deflection Measuring Equipment

Static deflection measuring devices include the plate bearing, curvature meter, Benkelman beam, the Traveling Deflectometer, and the La Croix Deflectorgraph. The Benkelman beam device produces the fundamental measurement on which are based most of the pavement design procedures developed in the last two decades. Consequently, its measurements can be applied directly to these existing empirical design procedures and it must be considered the most basic design survey device. It does have inaccuracies which become more serious as the pavement becomes stiffer. The plate bearing and curvature meter tests do not produce any more useful information than does the Benkelman beam and their data must be manipulated in some way (elastic analysis, empirical correlations) to produce corresponding Benkelman beam data. The deflectometer and deflectograph are mass inventory devices capable of traveling 4 and 9 miles a day respectively. Their basins are measured relative to reference bases that are influenced by the deflection basin. The deflections can be analyzed to give the elastic moduli of two layers but the results are more questionable than those which can be calculated from basins measured with steady state dynamic deflection devices. In addition, the mass of data that can be produced in a single day is more voluminous than is required for a design survey. For economy of data collection and processing effort, a statistical sampling survey is to be preferred to these mass inventory techniques.

Steady state dynamic deflection measuring devices include the Dynaflect and the Road Rater, both of which are available commercially; other devices that are used in research, such as the Waterways Experiment Station 9-kip and 16-kip vibrators, the Civil Engineering Research Facility 6.75-kip vibrator, the Shell 4-kip vibrator; and other devices in the developmental stage such as the Cox and Sons device currently being tested in Contra Costa County, California. The latter has an MTS loading apparatus placed in a traveling van.

The major conclusions reached in the study of these devices reported in Appendix B is as follows:

1. Surface deflection measurements must be made at more than one point on the pavement surface if the effect of various layers or their material properties is to be estimated.
2. Low frequency excitation, below about 10 HZ can be expected to produce deflection basins that are virtually identical with static basins. Furthermore, they can be analyzed to determine the elastic moduli of two layers. If unexpected results are noted in the low-frequency range, these may be due to the non-linear response of the electronics used in measuring the data.
3. These dynamic deflection devices mainly measure the effect of the subgrade on the overall deflection pattern. None of these devices can produce data which accurately represent the influence of the thickness or material properties of the surface course. The surface course elastic modulus can be calculated from the basin measurements but it is much less reliable than the calculated modulus of the subgrade.
4. The major effect of varying the frequency of excitation is to gain an appreciation of the viscoelastic response of the pavement structure. Trial-and-error calculations with a viscoelastic finite element computer program may be useful in estimating the viscoelastic properties of the <u>subgrade</u> mainly. Changes in the thickness and material properties of the surface courses will alter the resonant frequency of the pavement.

5. As noted in Chapter I and again in Appendix B, most pavement structures deflect reasonably linearly with load even when the loads reach 16-kips. Consequently, the assumption of linearity reasonably represents actual field behavior even under heavy loads. Naturally, there will be exceptions to this rule as in the case of pavements overlying moisture-sensitive soils in the spring break-up period. Thus, it will usually be the case that any device which applies a lighter load will produce results that may be extrapolated with reasonable confidence to heavy loads.
6. As a consequence of 2 and 5, it can be expected that any dynamic deflection device excited at low-frequencies and within the range of linear response of the electronic measurement equipment can be expected to produce reliable correlations with Benkelman beam measurements. Excitation at higher frequencies may not give deflections which correlate as well because the size of these deflections will vary with the viscoelastic properties of the pavement layers.
7. There is a need to make a better determination of the surface course material properties, both elastic and viscoelastic.
8. There is a need for a simpler method of determining by analysis the viscoelastic properties of at least one layer and preferably more.

In view of 1, 2, 5 and 6, it appears that a dynamic deflection device with a light load cycled at low frequency with motion sensors at several locations on the pavement surface will give the simplicity of operation, (light load), repeatability and analyzability of data, and the reliability of correlation with Benkelman beam data that was desired of design survey equipment. Ideally, this equipment should be highly mobile so that it can be used in a statistical sampling survey period; However, this equipment cannot, by itself, satisfy the needs noted under 7 and 8. More will be said of these points in Chapter V where the most promising pavement evaluation systems will be described in some detail.

Modulus Measuring Equipment

This unusual title has been adopted for this section to describe those devices which must rely on their ability to measure pavement moduli in order eventually to calculate deflections to correlate with Benkelman beam data. Included in this category are the wave propagation techniques, impulse and impedance methods, and, in general, any "transfer function" method. All of these methods except the transfer function are described in some detail in Appendix B.

A transfer function is defined as the frequency dependent ratio of the Fourier transforms of the output response and the input "signal" of a linear system (21). As an example, say that f(t) is the force input to a linear pavement system and x(t) is the measured displacement response of that system at a point. The displacement transfer function $T_x(\omega)$ is

$$T_x(\omega) = \frac{X(\omega)}{F(\omega)}$$

where

$$X(\omega) = \int_{t=-\infty}^{t=\infty} x(t)\ \exp(-j\omega t)dt,$$

the Fourier transform of x(t) and

$$F(\omega) = \int_{t=-\infty}^{t=\infty} f(t)\ \exp(-j\omega t)dt$$

the Fourier transform of f(t).

Similarly, the velocity transfer function $T_v(\omega)$ is

$$T_v(\omega) = \frac{V(\omega)}{F(\omega)} = \frac{1}{Z(\omega)}$$

where $V(\omega)$ is the Fourier transform of $v(t)$ and $Z(\omega)$ is the impedance function described in an earlier section of this chapter. Since the impedance function is really a special form of transfer function, all of the discussion presented and conclusions reached concerning impedance functions and their usefulness to pavement evaluation applies to <u>all</u> forms of transfer function.

Neither impedance methods nor any transfer function method in general can be expected to produce material properties of pavement layers except possibly through some form of empirical correlation. Changes of the transfer function with time may certainly indicate a deterioration of the pavement structure and despite its value as an indicator, it may not be used directly in design. This is the case with all presently available impulse load methods. Until an inexpensive method of analysis of impulsive loading of layered pavement systems is developed, the impulse methods may not prove useful to design. The use of electronic hardware to perform the fast Fourier Transform (45) on the input and response signals may provide the required analysis.

There are a wide variety of wave propagation methods many of which are described in Appendix B. A more thorough review of this equipment is found in a report by Watkins, Monismith, and Lysmer (22). Overall, the field experience with this type of equipment has been disappointing and this stems mainly from the difficulty in interpreting the data and extracting layer moduli. Improved methods of analysis of layered pavements are now available as a result of (22), but even this analysis shows that for accurate interpretations to be made, there must be a sharp contrast between the moduli on each side of an interface. This is a condition that rarely exists in the field except between the surface and base courses. Thus, it must be concluded that despite the encouraging recent developments in equipment and analysis methods, non-destructive wave propagation methods may not prove useful as design survey equipment in the near future.

The exception to this may be those devices which measure the modulus of the surface layer alone. This will provide valuable information to supplement steady-state dynamic deflection data. It may be that if the modulus of the top layer is known by independent means, the elastic moduli of two more layers may be determined from deflection data with a fairly simple computer program similar to the one presently used to determine the elastic moduli of two layers.

Crack Measuring Equipment

The same devices mentioned under decision surveys may be applicable in design surveys as well:

1. GM profilometer with a crack counting filter.
2. Dynaflect with an impulsive loading device.
3. Mobile acoustic crack detector using an electromagnetic tapper and two fluid-filled rolling wheels with microphones mounted in them.

The advantages and disadvantages of each of these systems have been described previously.

CONCLUSIONS

A statistical sampling survey of pavement condition is to be preferred over mass inventory methods because of its greater speed. Deflections are best measured with mobile dynamic deflection equipment which excites the pavement at frequencies below 10 HZ. Deflections measured by this method are nearly the same as static deflections and are the only deflections that can be expected theoretically to correlate well with Benkelman beam data. If deflections are measured at several points along the surface of a pavement, the same device used for the decision survey may also be used for a design survey, since its deflection basin data may be analyzed to determine layer moduli. All of the available dynamic deflection apparati measure subgrade properties with greater accuracy than those of the surface course. There is a need to do a better job of measuring the elastic and viscoelastic moduli of the surface course since it is the one layer on which the various rehabilitation techniques can be most readily applied.

The cracked condition of the pavement needs to be determined, primarily as an indicator of remaining pavement life but secondarily, to provide input data to overlay design schemes that use crack spacing as a factor. Three crack detection methods were proposed and one was tried out. The GM profilometer appears capable of detecting cracks some of which may yet be unseen. The other two methods are still conceptual although in principle they have both been tried out and found successful for other purposes: the Dynaflect with an impulsive loading device and the Mobile Acoustic Crack Detector.

The best of these methods from highway technology will be combined with the best from outside the highway field in Chapter V which describes the most promising pavement evaluation systems.

CHAPTER IV

TECHNOLOGY OUTSIDE THE HIGHWAY FIELD

A wide variety of non-destructive testing equipment is available in technological fields not related to highways. They can be divided roughly into ten classes:

1. Liquid penetrants
2. X-ray and associated radiographic techniques
3. Strain gaging
4. Leakage testing (outflow under pressure)
5. Magnetic and eddy - current testing
6. Optical scanning
7. Infrared scanning
8. Microwave sensing
9. Sonic and ultrasonic techniques
10. Fringe pattern and photoelastic techniques.

Several of these techniques can be discarded for highway uses by the nature of the materials, surfaces, and safety required for their operation. For example, liquid penetrants are used in industry to indicate cracks but they require a smooth surface texture to give adequate contrast between the crack and the intact material. The surface texture of pavements is too irregular and the absorption of the penetrant into the surface texture would mask out all but the very worst cracks, which are clearly visible to the eye. In any case, this would offer no advantage over a straight visual assessment of cracking. The use of x-ray or any of the associated radiographic techniques can be discarded for highway use because of the safety hazard and the technical problems encountered in their use as field testing methods. Strain gages are used in laboratory testing of concrete and asphalt but they do not lend themselves to field testing. Leakage testing using air or liquid under pressure can be discarded because of the kind of equipment that would be required and the relatively slow speed of operation. Magnetic and eddy current testing require a magnetic material for their operation, a property possessed by none of the highway materials. Fringe pattern and photoelastic techniques can be discarded because of the time and surface characteristics required for accurate testing. Once these techniques are eliminated, only four basic types of testing are left:

1. Optical scanning
2. Infrared scanning
3. Microwave sensing
4. Sonic and ultrasonic techniques

The specific methods investigated in this study are discussed below under the two categories of survey for which they would be best suited: rapid or decision surveys and detailed or design surveys.

Decision or Rapid Surveys

Martin Tracker - Optical scanning
Infrared thermometer microscope - Infrared scanning
Acoustic wave propagation - Sonic technique
Microwave sensing - Microwave sensing

Design or Detailed Surveys

Acoustic Holography - Sonic and ultrasonic technique
Vibroseis - Sonic technique
Duomorph - Strain-gaged piezoelectric crystal.

The duomorph is the only piece of equipment which does not fit any of the ten categories mentioned because of its unusual mode of operation. It is a promising technique that will be discussed in detail later in this chapter.

RAPID SURVEY EQUIPMENT

Martin Tracker

The Martin Tracker was designed initially as an optical displacement transducer (23). By focusing a black and white pattern into an array of photo diodes and comparing the output of adjacent diodes, the displacement of the pattern can be measured with an accuracy better than $\pm$ .001". In preliminary tests, it was found that with no modification the Martin Tracker could detect the presence of a pencil mark on a piece of white paper. Calculations showed that with the 5 microsecond response time of the photo diodes, the Tracker could give more than adequate output from cracks 1/16" wide while traveling at highway speeds. However, technical problems were encountered because the tracker could not distinquish between a crack and a piece of aggregate. Several different illumination schemes were considered, even the use of a laser, but the result was the same. The contrast between the asphalt and aggregate gave an output equal to or greater than that of a crack. This scheme seems suited for counting cracks on concrete pavements but not on asphalt pavements.

Infrared Scanning

Since infrared sensing equipment is not sensitive to the visible contrast of objects, the possibility of using an infrared thermometer was considered. The infrared thermometer is sensitive only to far infrared emission, which is basically a function of temperature and emissivity, which, in turn, is a property of material and surface condition. Tests were run using a Barnes RM-2A infrared microscope. The spectral range of wavelengths detected by this device is 1.8 to 5.5 microns. The band width of 20 KHz is more than adequate to detect 1/16" cracks at 60 mph. The system seemed to operate quite well. However, a crack could not be detected unless the temperature differential was greater than $2^{\circ}C$ between the crack and the adjacent pavement surface. Figure 4.1 shows the test setup used. The microscope head was mounted on an X-Y Positioner. It was scanned manually in the Y direction and was scanned in the X direction with a reversible electric motor. The output was recorded on an X-Y Plotter. Figure 4.2 shows a typical output from the infrared microscope scanning across a 0.15 inch crack.

In the laboratory, a 1000 watt flood lamp was used to simulate the solar heating of a pavement surface. During the period the RM-2A was on loan from the Barnes Company for evaluation, the weather prohibited any evaluation of the device on actual pavement. At the present state of the art the infrared detector doesn't seem to be suited for use as a rapid crack detector.

Vibration System (Wave Propagation)

One technique which appears promising for detecting the overall stiffness of the pavement system is a device which would record the phase, velocity and amplitude of a signal propagated through the pavement. In this technique a series of shaped input pulses, containing wave components in the range of 5-500 Hz, would be continuously coupled to the pavement as the test vehicle travels down the roadway. Receivers would be mounted both ahead of and behind the input point. The signal from each receiver would be averaged over several input pulses and the amplitude, velocity and phase of each

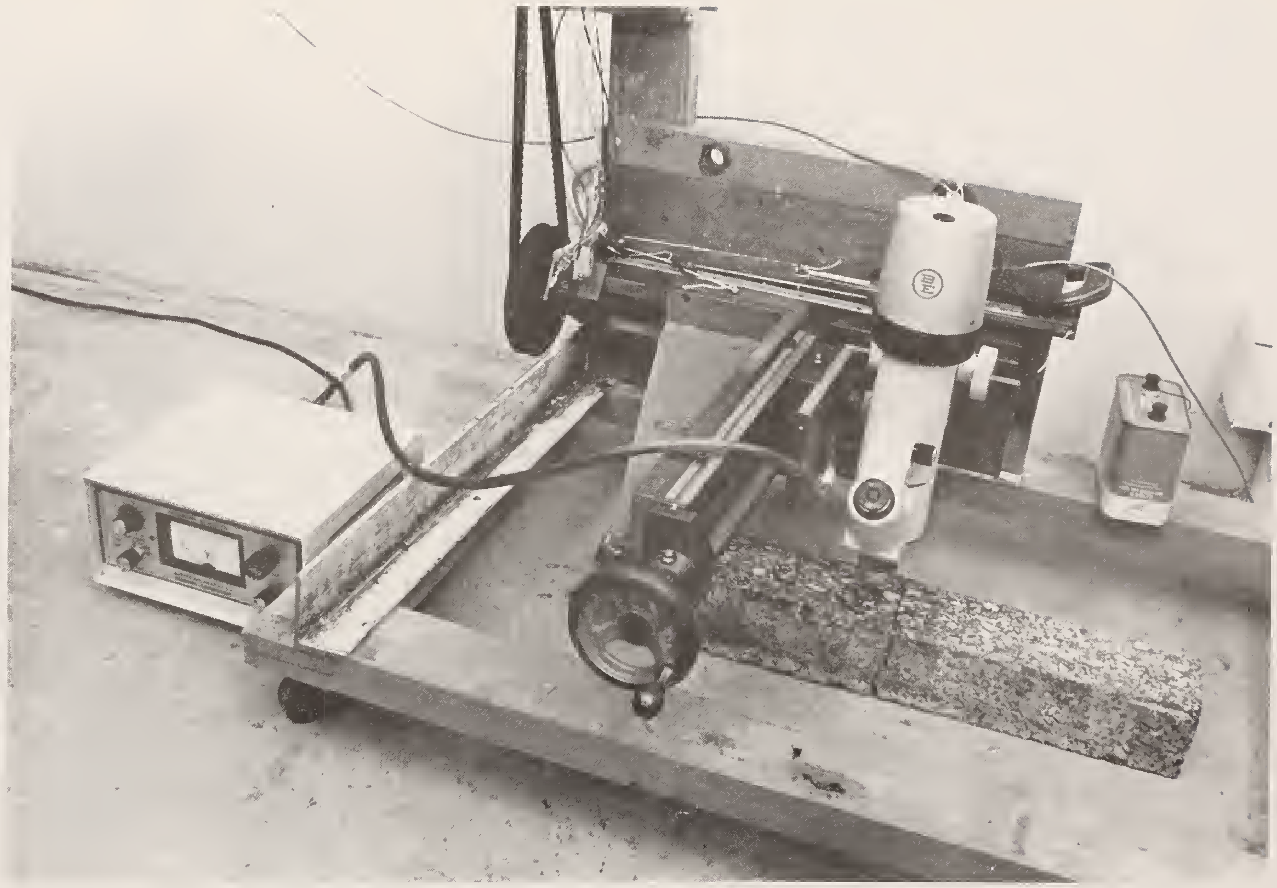


FIG. 4.1 TEST SET UP FOR INFRARED MICROSCOPE SCANNER

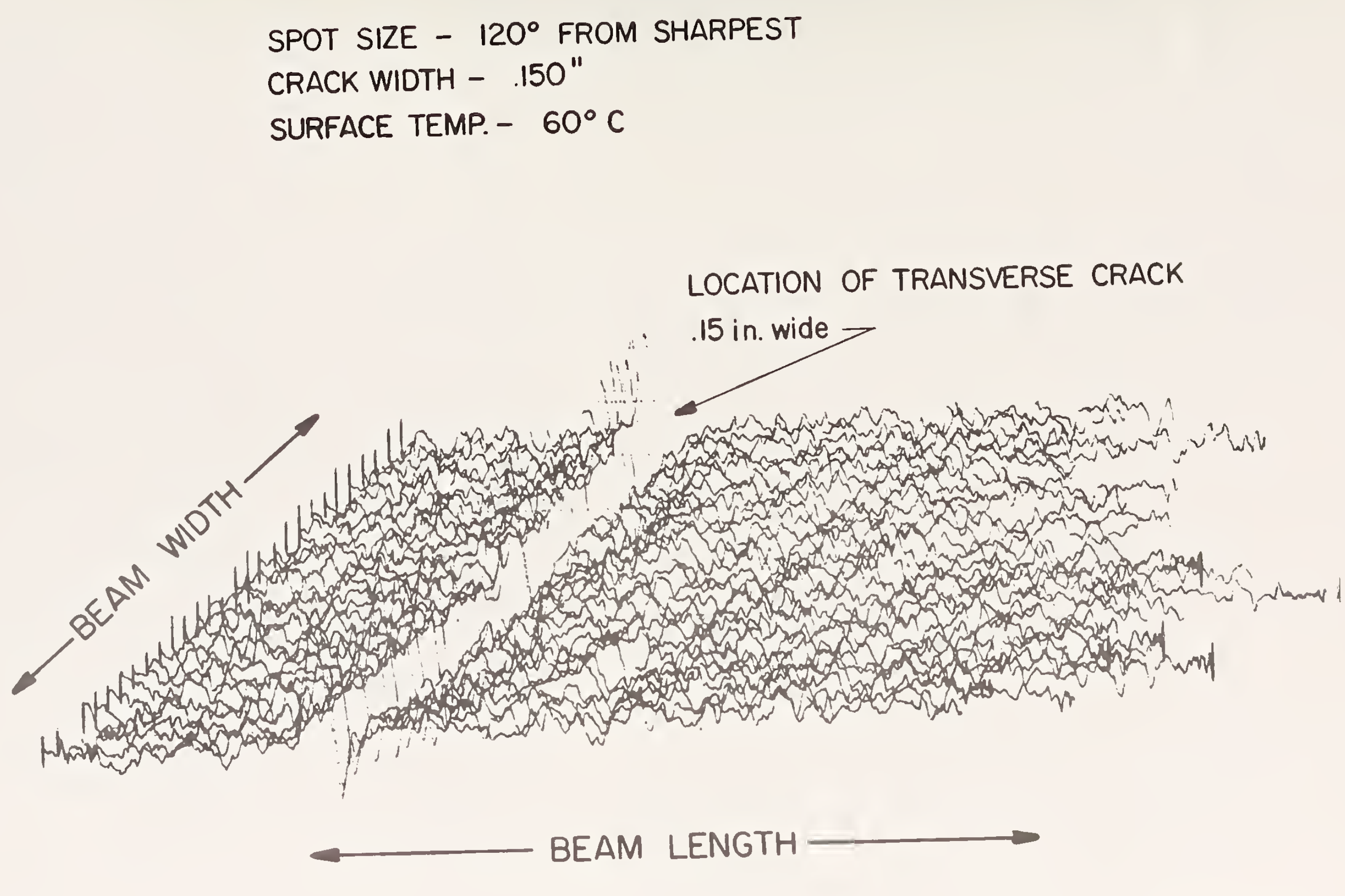


FIG. 4.2 OUTPUT FROM THE INFRARED MICROSCOPE SCANNING ACROSS A 0.15 IN. CRACK

frequency component of the composite input pulse would be analyzed. Although a system of the sort does not now exist, Holosonics Corp of Richland, Washington has used a similar stationary device for determining changes to the structure of the perma-frost along route of the Alaskan Pipeline. The system built by Holosonics slowly varied the frequency to a hydraulic pulser. Sensors were placed ahead of and behind the pulser. As the frequency was varied, the input force and the velocity and phase of the received signal were recorded. Different perma-frost structures (i.e. gravel, silt, clay) yielded different frequency/force-velocity ratio plots. A somewhat similar device has been developed for seismic purposes by Vibroseis, Inc. (24, 25). The interpretation and analysis of the output data can be a monumental task and at present, it is not certain precisely what pavement material properties might be extracted from such an analysis. This technique must be classified as a promising but unproven technique for highway purposes.

Microwave

The frequency band of microwaves is generally considered to be from 300 MHz to 300 GHz, corresponding to wavelengths of 1 meter to 1 mm.

Since World War II, microwaves have been used more and more for material testing. Microwaves were first used to test such components as waveguides, attenuators, resonant cavities, antennas and antenna covers. In the late 1950's microwave testing began to be used on materials not associated with radar equipment. The first use was to detect moisture concentrations on dielectric materials. Soon after, the thickness of metallic coatings on nonmetalic substrates and the thickness of dielectric slabs were measured. The measurement of thickness was followed by detecting voids, delamination and inclusions. Success in those areas led to the detection of chemical changes such as polymerization, oxidation, esterification, distillation and vulcanization. (Ref. 26).

Of primary concern to pavement applications is the measurement of moisture concentration, cracks, voids, inclusions and gradual variations in porosity and composition. It has been established that microwaves can be used to determine the moisture content of soils (Ref. 27, 28). A microwave system that could scan the pavement, penetrate the surface materials, and determine the average moisture content of the layers of base course and subgrade beneath, would be valuable in locating possible trouble areas for rutting and fatigue failures. The construction of such an instrument appears to be a real and practical possibility. There are, however, some problems associated with such a device. The ability of microwave to penetrate the surface layers has not been satisfactorily demonstrated and some means of isolating the effects of moisture from those of density and structure change would have to be devised.

Therefore, until these problems are overcome the use of microwave will have to be classified with the other possible but unproven techniques.

DESIGN SURVEY EQUIPMENT

Acoustic Holography

The field of scanned acoustic holography has made great strides in recent years (29, 30, 31, 32, 33). Most of the advances have been in the areas of nondestructive testing and bio-medical imaging. The technique has been considered for use in detecting cracks in the face of a concrete dam and to investigate the effect of supertransport plane loads on the structure of runways. However, no one has to date actually spent any money to develop these systems. According to the technical personnel at Holosonics, the development of a system to image pavement structure is a logical extension of present equipment capabilities. The only major changes would be the decrease in illuminating frequency and a corresponding increase in scanned area. The system they proposed as a practical means of observing cracking in pavement consisted of an illuminating frequency of 40 KHz and a minimum scanning area of 7' x 7'. This system, it was generally agreed, would image cracks in the asphalt structure of .04" width. The cracks may either be visible at the surface or as yet invisible. A scanned holographic system for observing pavement cracking would include all of the presently available options, including the 3-D display which is a very effective way of displaying the internal structure of a material because it allows the observer to rotate the image about the horizontal and vertical axis and thus change the point of viewing of the observer.

The system would also include the time-gated return function which allows the observer to look at defects in a particular plane within the sample by gating out all signal and noise above and below that plane.

The possible uses of this sort of system are numerous. One use would be to compare cracking patterns of a section of pavement before and after the application of a test load. This would allow graphical analysis of the increase in cracking (both number and severity) due to the load application, thus providing a way of checking design. It would also be valuable in observing environmental deterioration by looking at crack patterns as they change with the seasons on a test section exposed only to environmental effects. It could also be used to monitor sections of pavement subjected to both traffic and severe environmental conditions (i.e., freeze-thaw)

This is a very promising method that has been proven in other fields and could be fairly readily adapted to highway applications.

Duomorph

The duomorph has been used in the rocket industry to monitor the changing modulus of solid rocket propellent (32). The duomorph sensor, designed for dynamic testing, consists of a thin disc of metal (brass or stainless steel) with a PZT crystal cemented to each side. A strain gage is cemented to each face of the device.

Each outside face of each crystal is held at ground potential and AC voltage is applied to the metal shim between them. The AC voltage causes one crystal to expand and the other to contract radially producing a bending. The strain gages record the amount of bending. Figure 4.3 shows a cross section and plan view of a typical duomorph device and Fig. 4.4 is a photograph of the duomorph which was built and pilot-tested on asphalt concrete to determine its ease of application

The strength of the duomorph can be controlled by changing the thickness to diameter ratio of the crystals. This permits the construction of sensors which are sensitive to different ranges of stiffness of materials with which they are brought into intimate contact. The dynamic range of the duomorph device varies from DC to 4000 Hz depending on the thickness-to-diameter ratio and the stiffness of the surrounding material.

Basically, the analysis of the interaction between the duomorph and the surrounding material is based on the difference in bending of the duomorph in air and in the material under test. The difference in total amplitude and phase of the strain gage output are inputs to an existing computer code (35). The output of the computer code includes the real and imaginary parts of the viscoelastic complex modulus, E' and E", of the material under test.

The approach currently used to obtain amplitude-phase changes from air to the test material is an oscilloscope photograph where the applied voltage drives the horizontal axis and the deflection strain drives the vertical axis. The resulting ellipse is measured with dividers by hand and the results are punched on a computer card for input to the computer program.

It is felt that the amount of information available could be increased and the analysis simplified by using a variable AC frequency scanned at a constant rate over a band from 2-2000 Hz with the deflection amplitude and phase plotted on an X-Y-Y Plotter. This approach would yield information about the response of a material over a range of frequencies in the same time required now to get one single data point. A block diagram of such a system is shown in Fig. 4.5. The relationship between frequency, phase and amplitude as illustrated in Fig. 4.6 is needed to accurately classify the viscoelastic properties of a material. The viscoelastic properties of asphalt surface or overlay materials would be very helpful in predicting crack growth rate.

The major advantages of the duomorph device as a test instrument are simplicity and repeatability. The duomorph is simply placed on a sample to be evaluated so that continuous contact is assured and the output is read directly from either a digital readout or a continuous graph plotted automatically. Analysis can then be done easily by a programable calculator similar to the Hewlett-Packard model 9821A. Repeatability and reproducibility are assured by the removal of any subjective evaluation judgment on the part of the operator. The system can be periodically checked out to

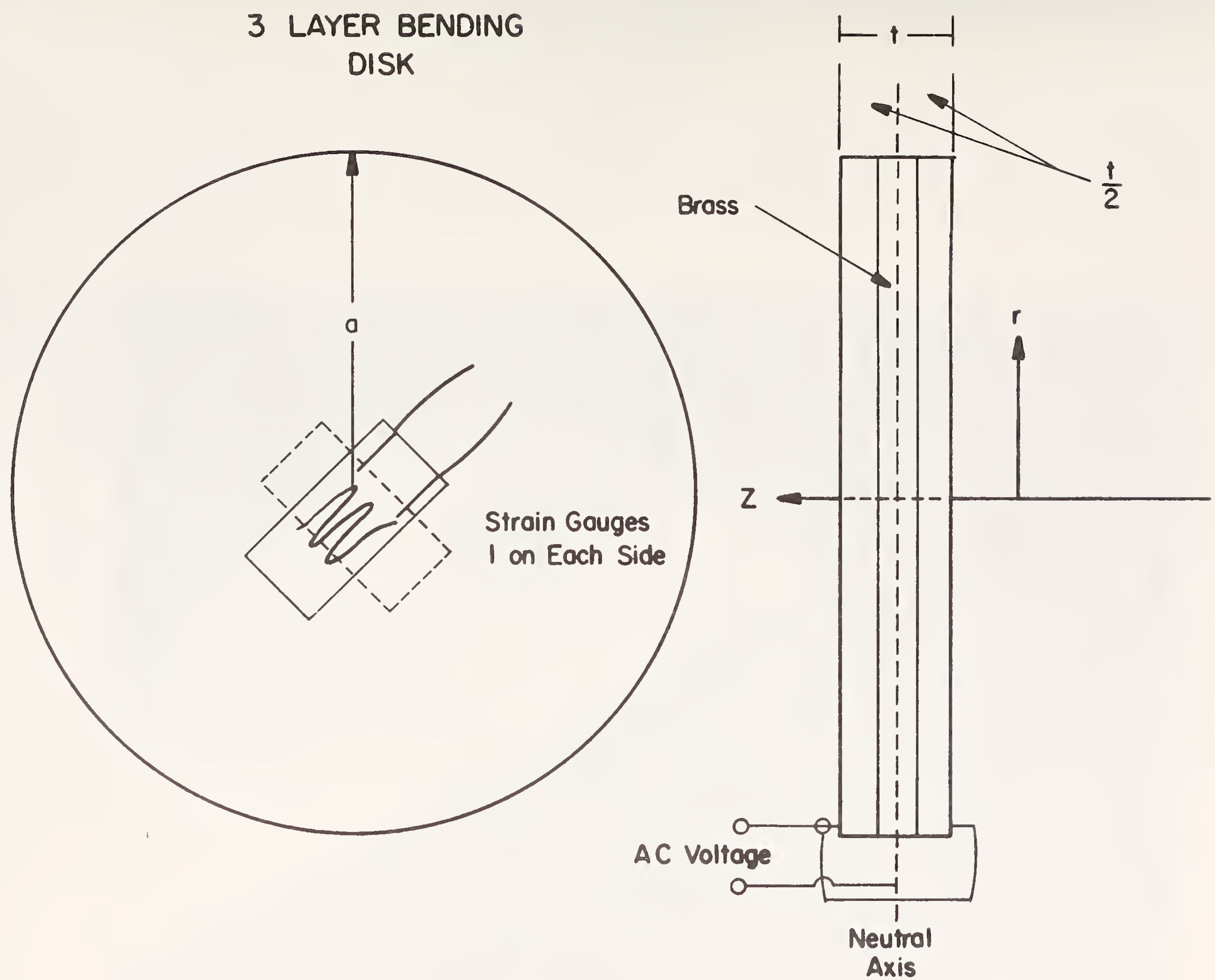


FIG. 4.3 CROSS SECTION AND PLAN VIEW OF THE DUOMORPH

FIG. 4.4 THE DUOMORPH

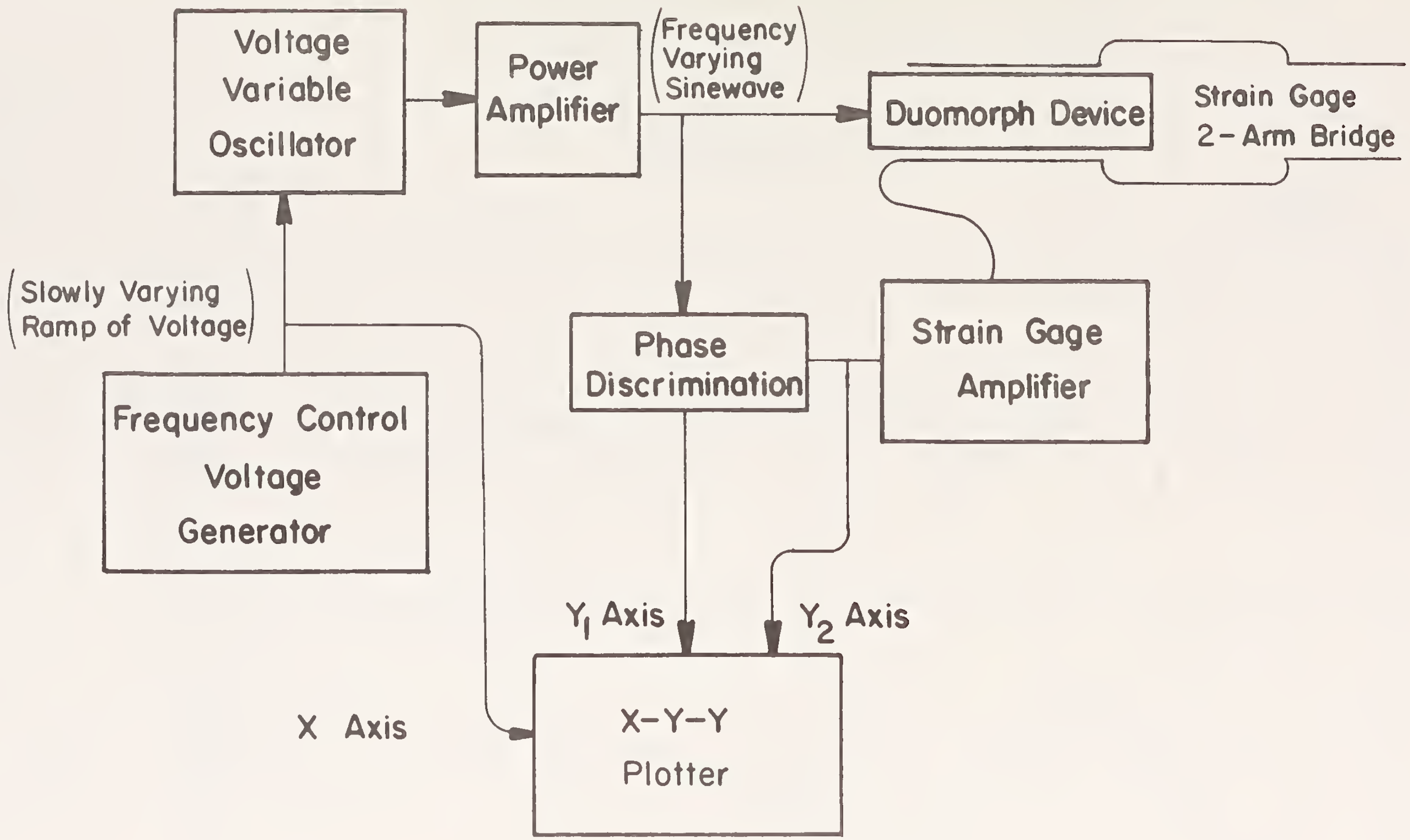


FIG. 4.5 BLOCK DIAGRAM OF A DUOMORPH DATA ACQUISITION SYSTEM

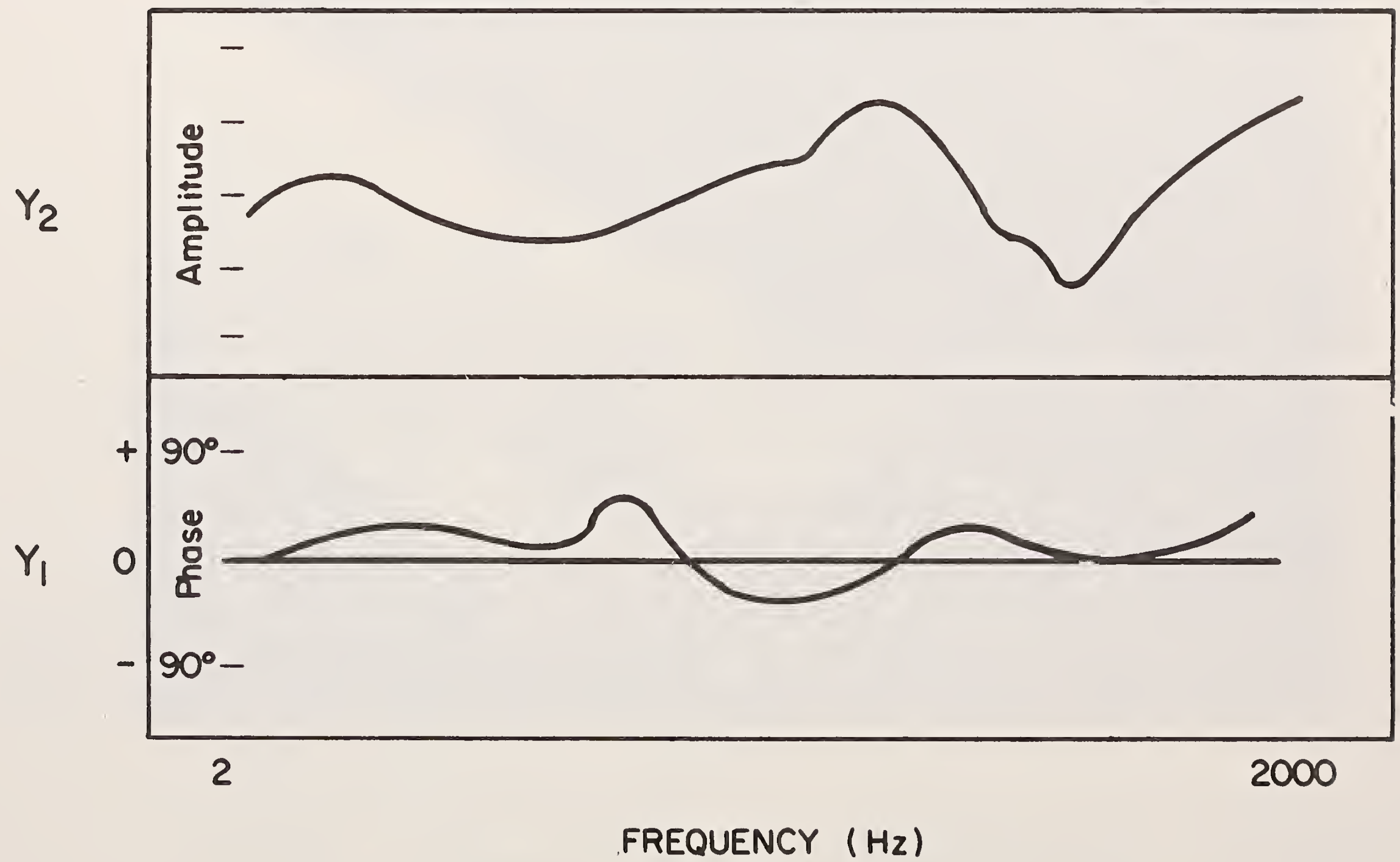


FIG. 4.6 SCHEMATIC OUTPUT FROM DUOMORPH MEASUREMENTS

assure proper operation by using standard viscosity silicone fluids and newly developed standard viscoelastic rubber samples for calibration tests.

One added advantage is that the same concept, even the same test tool, can be used in the field or in the laboratory for quality control testing. The major use of the duomorph in evaluating pavements is in determining the elastic and viscoelastic properties of the pavement surface materials. The properties may then be used in the design of overlays and in the prediction of crack growth rates in the existing pavement. This is a promising equipment concept that appears to be readily adaptible to highway applications.

CONCLUSION

From the review of industrial nondestructive testing procedures there appears to be three which merit more detailed study. They are:

1. Acoustic holography
2. Vibration methods
3. Duomorph

The holographic technique is an adaptation of a currently operational system. It will, however, require 4 to 6 man years to complete the adaptation and proof-test the prototype equipment.

Due to the enormous amount of data analysis required for the vibration technique and the fact that very little work has been done toward the theoretical determination of the elastic moduli of multilayered systems by this technique, it is felt that a minimum of 6 man years would be required to fully develop and analyze a system of this sort.

The duomorph has been used in the solid rocket propellant field. The theoretical analysis of a duomorph embedded in a material has been done and a computer code has been written to handle data reduction. Most of the problems initially encountered in development of the duomorph sensor have been overcome, and calculations indicate the operation of the duomorph should extend to materials having a stiffness greater than that of flexible pavement.

CHAPTER V

THE MOST PROMISING PAVEMENT EVALUATION SYSTEMS

The development of equipment for pavement evaluation should have the objective of providing two major kinds of data that are needed by highway agencies:

1. Statistically reliable distress data from all sections of a roadway network for the purpose of making planning, budgetary, and rehabilitation decisions.
2. Reliable data which accurately represent the structural condition of the pavement and may be used for design. These data may be related both to the response of the pavement materials to applied load and non-load stresses and to the limiting condition of these materials. These data may be used to design either new pavements or overlays.

The ideal piece of equipment would be able to furnish both kinds of data but at the present stage, no such equipment appears to be feasible. The desirable characteristics of such equipment include speed of operation, repeatability, its ability to produce data that can be correlated with previous experience, and its ability to be analyzed to produce material properties or limiting conditions for use in presently developing design procedures.

Speed of Operation. It appears that there are two major reasons for having equipment that can make rapid measurements:

1. More data can be collected quickly to provide mass inventory information for network decisions.

2. Personnel will not be exposed to traffic hazards nor will traffic flow be interrupted if equipment can be built to operate at highway speeds. This is most important in areas of high traffic volume.

As has been shown in this report, a massive amount of data may not need to be collected as long as a statistically significant number of samples are taken. Simple studies with existing apparatus may show several cases where a reduction can be made in the mass of data collected which will still retain about the same reliability in the estimates of pavement condition. This makes the speed of operation of a given device less important than other desirable characteristics.

The safety of the personnel operating the equipment is a matter of prime importance in choosing equipment for decision surveys. Consequently, in making these surveys on freeways or other high volume roads, emphasis should be placed on visual ratings and crack-counting with devices that are mounted on vehicles which travel at highway speeds. Design surveys should be conducted during offpeak hours or at night when safety hazards can be reduced without disrupting traffic flow.

These two considerations show that the speed of operation of measuring equipment should be considered as less important than the quality of information the equipment can provide.

Quality of Data. The repeatability of the measurements and their ability to be correlated with previous experience are equipment characteristics that need no explanation. The ability of the data to be analyzed to determine relevant material properties is an important feature mainly for design surveys.

The kinds of data that are required in current and developing <u>design</u> procedures which can be produced by the techniques considered in this report are as follows:

Elastic modulus

Viscoelastic moduli

Linear body influence coefficients

Surface deflections and curvatures

Crack spacing and severity

Crack size and location

The kinds of distress data for <u>decision surveys</u> that can be produced by non-destructive test equipment include only:

Crack spacing and severity

Rut depth

The surface characteristics of a pavement that can be measured include:

Skid resistance

Roughness

Texture

Some of these are measured on a mass inventory basis and others are measured at selected spots for design and research purposes. It is beyond the scope of this report to consider equipment for measuring surface characteristics.

The quality of these data depends upon two factors: the precision of the equipment and the analysis of the data. Both of these have been considered in previous chapters and they will be reviewed briefly for each of the kinds of data that was mentioned above.

ELASTIC MODULI

At the present time, the best developed methods of analysis are those associated with the surface deflection devices although these can determine the material properties for only two layers. Wave propagation methods show promise, but the analysis of the data works well only if there is a sharp distinction between the elastic moduli of adjacent layers. Impulse methods require the collection and analysis of large quantities of data either by analog or digital computer techniques. If efficient methods of Fourier analysis of these data can be devised, then these methods may eventually produce data from which elastic moduli of each layer may be calculated. Elastic moduli--or pseudo-elastic moduli such as the stiffness coefficients derived from Dynaflect data for the Texas Flexible Pavement Design System--are used in design methods for both pavements and overlays in which linearly elastic theory is being used more frequently than ever before.

VISCOELASTIC MODULI

A number of methods are available for determining field values of viscoelastic moduli. In some cases the equipment has been developed and only needs modification to produce useable data. In other cases, the analysis methods have been developed or can be developed fairly easily but the equipment is not available. The situation as it stands at present is reviewed below.

Dynamic Deflection Equipment. If viscoelastic moduli are to be measured with dynamic deflection equipment, it must be capable of measuring surface deflections accurately at several frequencies, one of which is well below the resonant frequency and another of which is near or greater than the resonant frequency. The applied loading should be an accurate sinusoidal load. A linear viscoelastic finite element technique such as demonstrated in Appendix B may then be used to determine approximately the real and imaginary parts of the complex modulus. Because of the approximation involved and the uncertainty of instrumental error, it is believed that only the viscoelastic properties of the subgrade can be determined with some confidence with this method. Thus, approximate viscoelastic properties can be derived from the results of dynamic deflection tests made at several frequencies.

Creep Tests. Perhaps a more promising technique from the theoretical point of view is a field creep test in which a load is applied suddenly and surface deflections away from the load are measured with time. It is not certain that equipment is available which could measure the long-term surface deflections with enough accuracy. Such equipment would have to maintain a stable reference line that is accurate within a mil/(0.001 inch) so that creep deflections can be measured with that accuracy. The feet of the instrument which establishes the reference line must be sufficiently removed from the loading point so that its stability will be unaffected by creep in the basin. A laser reference line may prove to be suitable for this purpose. The geometry of the load must remain very simple and approach a point load if the analysis is to produce valid results. The required geometry of the load is discussed more in detail in Appendix B.

If all of these restrictions are met, then the analysis can, at least theoretically, derive the viscoelastic properties of two layers by combining Scrivner's equation (15) with approximate Laplace transform theory developed by Schapery (36) and with an efficient multi-dimensional search method. In

In principle, the procedure is as follows. The equation for the deflection $w(t)$ at distance r for a point load P is

$$\frac{4\pi r}{3P}\ \frac{w(t)}{D_1(t)} = \{s[1 + \int_0^{10\frac{r}{h}} (V - 1)\ J_0(x)\ dx]\}\ s = \frac{1}{2t} \qquad (5.1)$$

where h = the thickness of the top layer

$x = m\frac{r}{h}$

$D_1(t)$ = the creep compliance of the first layer at time t.

$= [D_{1_1} + D_{11_1}\ \Gamma(1 + n_1)\ (2t)^{n_1}]$

$\Gamma(1 + n_1)$ = the gamma function of argument $(1 + n_1)$

D_{1_1} = the reciprocal of the elastic modulus of layer 1 which can be derived from analysis of Dynaflect data.

D_{11_1} = a creep constant for the first layer.

n_1 = the log-log slope of the creep compliance versus time curve for layer 1.

$$\tilde{V} = \frac{1 + 4\tilde{N}\ me^{-2m} - \tilde{N}^2\ e^{-4m}}{1 - 2\tilde{N}(1+2m^2)e^{-2m} + \tilde{N}^2 e^{-4m}}$$

$$\tilde{N} = \frac{a - 1}{a + 1}$$

$$a = [\frac{b + c\ \Gamma(1+n_2)\ s^{-n2}}{1 + d\ \Gamma(1+n_1)\ s^{-n_1}}] \quad s = \frac{1}{2t}$$

$b = \frac{D_{1_2}}{D_{1_1}} = \frac{E_1}{E_2}$ which is determined by analysis of Dynaflect data

$c = \frac{D_{11_2}}{D_{1_1}}$, a second-layer creep constant which must be determined by analysis.

$$d = \frac{D_{11_1}}{D_{1_1}}$$, a first layer creep constant which must be determined.

n_2 = the log-log slope of the creep compliance-time curve for layer 2.

The log-log slopes n_1 and n_2 will usually be between 0 and 1. The trial-and-error analysis will proceed as follows.

1. Assume initial values of c, d, n_1, and n_2.
2. Calculate the theoretical deflection ratios, r.

$$r_{ij} = \frac{W_1(t_i)}{W_j(t_i)} \quad \text{for } i, j = 2, 3, 4 \text{ and } 5$$

   This means that deflections at five locations at four separate times must be calculated with Equation 5.1. W_1 is the deflection closest to the load.

3. Calculate the measured deflection ratios $\hat{r}$, at the same points and same times,

$$\hat{r}_{ij} = \frac{\hat{W}_1(t_i)}{\hat{W}_j(t_i)}$$

4. Form the sum of square errors for each time, t_i. This sum is denoted by f_i.

$$f_i = \sum_{j=2}^{5} (r_{ij} - \hat{r}_{ij})^2$$

5. By using an efficient multidimensional search technique, determine values for c, d, n_1, and n_2 which reduce each of the f_i to within a small tolerance of zero.

Whether or not the search technique will converge can only be proven by trial. Conceptually, if convergence can be assumed, then this method, combined with the Scrivner's (15) elastic analysis of Dynaflect data will produce viscoelastic properties of two layers. If a three-layer elastic analysis were developed, it could be used in this same way to determine the viscoelastic properties of three layers. Of course, the numerical difficulties and the number of computer calculations also begins to accumulate, making it more difficult to say whether such a determination is economically feasible. From the point of view of usefulness, simplicity, and relative accuracy of obtaining viscoelastic field data, the creep test combined with Dynaflect analysis is to be preferred to the trial-and-error procedure using dynamic deflection data at various frequencies. It should also be noted that the creep data measured by this method are not consistent with the complex modulus formulation of viscoelastic properties used in the dynamic deflection method which uses variable frequencies and is explained in detail in Appendix B. The only time when these two definitions of viscoelasticity become fairly consistent is when the "creep" strain greatly exceeds the "elastic" strain.

Duomorph Measurements. The principle of operation and analysis of duomorph data have been presented in Chapter 4. Since the duomorph is a small device, it is expected to give both elastic and viscoelastic moduli of the surface course alone. As such, it is a complement to the two previously mentioned tests which will produce information about subgrade more accurately than for any other layer.

Use of Viscoelastic Properties in Design. Viscoelastic properties are not used in most current design procedures. However, viscoelastic calculation forms a central element of the FHWA design system VESYS II which will probably be used much more widely in the future. There are several reasons for this, the main ones being associated with prediction of various forms of distress, specifically rutting, fatigue, thermal and shrinkage cracking. Rutting can be promoted by a mismatch of creep compliances between two adjacent layers. Fatigue properties are dependent upon the frequency of testing and consequently upon the frequency of loading applied by a passing stream of traffic. Single axle vehicles will apply loads at around 6 to 8 Hz while tandem axles will apply loading frequencies of perhaps 15 to 20 Hz. The frequency dependence of fatigue properties and the amplified response of pavements at 20 Hz or above shows the importance of viscous damping in prolonging the fatigue life of a pavement. Fatigue, thermal, and shrinkage cracking can all be explained theoretically by developments within the field of viscoelastic fracture mechanics. Schapery (38) has shown that the rate of crack propagation depends exponentially on the value of n - the slope of the log-creep compliance - log-time curve. The smaller n becomes, the faster cracks will propagate. This becomes very important in designing crack-resistant overlays.

Thus, it is expected that future design methods which account for distress explicitly will use viscoelastic properties more than they do now. A creep test with a stable reference line, accurate measurements with time, and the development of convergent analytical procedure will provide field data for future viscoelastic design calculations.

LINEAR BODY INFLUENCE COEFFICIENTS

The design of overlays is accomplished by various means at present, most of which are either intuitive or empirical (39). However, there are some design methods which have been proposed recently which are based on elastic theory (40,41,42). Even these methods generally require an assessment by the designer of the quality of the subgrade or the condition of the existing surface, or both. It is both unlikely and undesirable that this element of experience and judgment should ever be removed from overlay design. In designing an overlay, it would be desirable, if possible, to treat the pavement as a single homogeneous layer with an "effective" modulus and design the overlay to provide enough thickness to reduce its own deflections and tensile stresses to an acceptable level. In some cases, notably the Virginia (42), and the U.S. Steel (41) methods, the existing pavement is treated as a multi-layered system. Another method is possible, by using field measurements alone, and by assuming that the pavement structure is linear, to design an overlay without considering the depth and modulus of any of the pavement layers. The method is called a Green's function analysis which uses measured influence coefficients, and is based on the reciprocal theorem of Betti and Rayleigh (43). Measured vertical and horizontal deflections of a pavement surface due to unit horizontal and vertical loads are the influence coefficients used in this method. When a load passes over a new overlay, it will generate forces and displacements at the boundary between the old and the new pavement. These forces $\bar{\underset{\sim}{t}}$ and displacements $\bar{\underset{\sim}{u}}$, which are situated at the nodal points of a finite element mesh, are related by a matrix of influence coefficients, A, which are derived from the field measurements.

$$\bar{\underset{\sim}{u}} = A\,\bar{\underset{\sim}{t}} \tag{5.3}$$

The finite element solution for the displacements of the overlay, which can include thermal stresses, is given by:

$$K\,\underset{\sim}{u} = \underset{\sim}{P} \tag{5.4}$$

in which K is the stiffness matrix, $\underset{\sim}{u}$ is the displacement vector, and $\underset{\sim}{P}$ is the force vector. The matrix K may be partitioned into that part related to interface displacements K_2 and those parts related to the other displacements, as shown below

$$\left[\begin{array}{c|c} K_1 & C^T \\ \hline C & K_2 \end{array}\right] \left\{\begin{array}{c} \underset{\sim}{u} \\ \hline \underset{\sim}{\bar{u}} \end{array}\right\} = \left\{\begin{array}{c} \underset{\sim}{P} \\ \underset{\sim}{\phi} \\ \hline \underset{\sim}{\bar{t}} \end{array}\right\} \tag{5.5}$$

The force vector is partitioned into the wheel forces, $\underset{\sim}{P}$, interface forces $\underset{\sim}{\bar{t}}$, and zero forces, $\underset{\sim}{\phi}$, which exist everywhere else. Since Eq. 5.3 gives

$$A^{-1}\,\underset{\sim}{\bar{u}} = \underset{\sim}{\bar{t}} \tag{5.6}$$

This may be subtracted from both sides of matrix equation 5.5 to give

$$\left[\begin{array}{c|c} K_1 & C^T \\ \hline C & K_2 - A^{-1} \end{array}\right] \left\{\begin{array}{c} \underset{\sim}{u} \\ \hline \underset{\sim}{\bar{u}} \end{array}\right\} = \left\{\begin{array}{c} \underset{\sim}{P} \\ \underset{\sim}{\phi} \end{array}\right\} \tag{5.7}$$

which can be solved by standard matrix inversion methods. The significant part of this is that the displacements and stresses in the overlay may be determined once the inverse of the matrix of influence coefficients is subtracted from the stiffness sub-matrix, K_2. The calculations would be simple and would require comparatively little computer time.

Field measurement of the influence coefficients would require both horizontal and vertical sinusoidal force generators to be applied separately to the pavement. While each of these forces is being applied, both horizontal and vertical displacements of the pavement surface need to be measured at several locations away from the applied loads in order to get an accurate distribution of the influence coefficients.

Geophones are available at present which can measure both horizontal and vertical displacements. A Dynaflect-type of device can generate the vertical force and another type of device would need to be developed to generate the horizontal force.

A pair of rotating eccentric weights mounted one above the other on the same vertical shaft can provide the desired horizontal load. The weights will have to be sized so that the instrument applies only horizontal load and no moment at the surface of the pavement. In principle this is possible if the eccentricities of the two weights are arranged at 180 degrees from each other and the sum of their centrifugal forces times their respective heights above the pavement are equal to zero. The instrument design equations which specify the size and location of the eccentric masses m_1 and m_2 on the vertical shaft are as follows:

$F_1 d_1 = F_2 d_2$ (condition for zero moment at the pavement surface)

and $F_1 - F_2 = (m_1 r_1 - m_2 r_2)\omega^2$ (size of horizontal force)

where

$$F_1 = m_1 r_1 \omega^2$$

$$F_2 = m_2 r_2 \omega^2$$

d_1, d_2 = heights of m_1 and m_2 above the pavement surface

ω = speed of rotation radians/sec.

In addition to the problems involved in designing the equipment, the major field operation problem will be the positive attachment of the horizontal force generating device to the pavement.

The usefulness of this kind of device to overlay design procedures is apparent. Overlays with multiple layers of different materials having distinctively different elastic moduli may be considered, and design stresses may be calculated at any point within the overlay. It appears that many overlay design methods for cracked pavements, both rigid and flexible, are presently recommending the use of a stress-relieving or bond-breaking interface to prevent or reduce reflection cracking. Of course, when pavements are cracked, it is difficult to say whether the old pavement still responds to load as a linear body. In this case, linearity only becomes an assumption which must be verified by field measurements.

In view of the wide variety of materials that are presently in use in overlays and the wider variety that will be used in the future as a result of economic adjustments in the energy, petroleum, and mining industries, it appears unlikely that a reasonable set of design charts may be constructed to handle any more than the most popular combinations of materials. Thus, the development of the horizontal force generator and the overlay analysis technique using finite element analysis may prove to be cost effective and beneficial.

SURFACE DEFLECTIONS AND CURVATURES

All of the equipment which measures surface deflections for determining moduli of the pavement layers also produces information that can be used directly in the empirical design procedures that use either deflections or curvature of the pavement surface. Wave propagation or impulse techniques must first have their data analyzed to produce elastic moduli from which surface curvature or deflection may be calculated. Since all of these methods have been discussed elsewhere in this report, they will not be described any further here. Any piece of equipment that can measure only a single deflection or curvature cannot be expected to produce information that will be useful for pavement design methods that are being developed at present. Mass inventory equipment such as the California Traveling Deflectometer or the Lacroix Deflectograph which can measure accurate deflection basins and do produce information that may be used in these developing design procedures, are nevertheless not mobile enough to cover large mileages of roadway network and cannot produce information on the viscoelastic properties of the pavements layers. If either of these devices is used to sample the strength of representative pavement sections their daily mileage rate can be increased, and they can provide valuable information for <u>decision</u> surveys.

One device for measuring dynamic deflection basins under the passage of wheel loads traveling at highway speeds is the accelerometer device constructed by Swift (44). This device is imbedded in the pavement and records the surface deflection at a point as a given wheel load passes, thus tracing out a deflection basin. If the basin is symmetric about the point of greatest deflection, this indicates that the pavement is behaving elastically. If the basin is skewed, then it indicates viscoelastic behavior of the pavement. The latter has usually been found to be the case on all pavements. Up to the present, no method of analyzing these deflection basins to extract viscoelastic moduli has been devised. If such an analysis method can be developed, it can provide valuable information for <u>design</u> surveys.

CRACK SPACING AND SEVERITY

For measurement purposes, there are three basic types of cracks:

Longitudinal
Transverse
Map and alligator cracking

The length of longitudinal cracks can be estimated by eye and stop-watch while traveling at highway speeds. The presence of map or alligator cracking can be observed visually and the area covered by such cracking can be estimated with some degree of accuracy by visual observation. The only kind of cracking that cannot be observed accurately at highway speeds is transverse cracking.

Several methods have been presented in this report for measuring transverse cracking with sufficient accuracy for use in design:

Crack counting with the GM Profilometer

Dynaflect with an impulsive loading device to pick up cracks by attenuation between geophones

Mobile acoustic crack detector using an electromagnetic tapper to make the sound and fluid-filled wheels with microphones inside to pick up the sound.

Crack counting with an infrared thermometer microscope on flexible pavements.

Crack counting with a photon light beam counter (Martin Tracker) on concrete pavements.

Although the last two methods can count the number of cracks crossed in a given distance, they cannot measure the severity of the cracking or compile statistics on crack spacing and severity that can apparently be accomplished with the GM Profilometer. The latter device is expected to be a valuable tool in <u>design</u> surveys for calculating the thickness of overlays to prevent reflection cracking. However, there are some disadvantages to this method of crack detection.

The Profilometer data must be converted from analog to digital form and be read onto magnetic tape before it can be processed through the crack-counting filter. Unless the computer programs are written very efficiently this tape conversion and filtering can be somewhat time-consuming and expensive. The result is a very comprehensive picture of the transverse cracking pattern on a section of pavement.

A less comprehensive picture but one which can be obtained more quickly is the crack count from the Mobile Acoustic Crack Detector. Every time the signal attenuation between the two wheels drops below a pre-set level, a crack is counted. It may or may not be able to distinquish a large from a small crack or a visible crack from a hidden one. It can give the crack count immediately. If the distance traveled is measured by an odometer attached to one of the rolling wheels, the average crack spacing can be determined. The speed of operation and low cost of data acquisition makes this a promising pavement evaluation system.

CRACK SIZE AND LOCATION

Acoustic holography has proven to be potentially very useful as a tool for research and detailed investigations into the size and location of unseen cracks. Information gained by observing fatigue, shrinkage, and thermal cracks in the field can give valuable field data on the cracking performance of pavements. Development of this equipment could prove to be one of the most valuable additions to pavement evaluation equipment that could be made, especially if field data on crack growth can be measured.

DECISION SURVEYS

Some pavement evaluation equipment can measure cracking and rutting data for decision surveys. Both the infrared thermometer microscope and the Martin Tracker can count cracks rapidly and several simple devices can sample rut depth measurements with sufficient accuracy and frequency to be adequate for decision surveys. The mobile acoustic crack detector in conjunction with a Dynaflect device may be used in a statistical sampling survey to provide most of the measured data required for the purposes of a decision survey.

CONCLUSIONS

This chapter has enumerated the most promising pavement evaluation methods that are currently available or within range of development in a reasonably short period of time. To qualify as a pavement evaluation method, not only must the equipment be able to measure data with precision and repeatability, but the data must be in a form that can be analyzed to produce material properties that are useful in current and developing pavement design procedures.

CHAPTER VI

RECOMMENDATIONS FOR FUTURE DEVELOPMENT

The recommendations in this chapter are broken into two parts. The first part includes recommended equipment and analytical developments for the second phase of research contract DOT-FH-11-8264, "Pavement Evaluation". The second part of the recommendations include those items of equipment and analysis which should be developed in the near future to aid in the processes of making rehabilitation decisions and designing pavements and overlays. When these items of equipment are developed, they are expected to provide substantial benefits, but they cannot be developed within the scope of the present contract.

CONTRACT DEVELOPMENT

The following developments are recommended:

1. The duomorph, including all of the associated equipment and analysis techniques.
2. The crack counting filter using GM profilometer data. This includes field verification on flexible and rigid pavement.
3. The mobile acoustic crack detector, including all equipment development and field verification on flexible and rigid pavement.

All three of these developments will require field verification of the prototype equipment.

FUTURE DEVELOPMENT

Recommended future developments include the following:

1. An acoustic holographic scanning apparatus which is capable of detecting unseen cracks and measuring their size and location.
2. A variable frequency device that applies a true sinusoidal loading to a pavement and measures surface deflections accurately at all frequencies. This device also requires the development of an inexpensive analysis technique.
3. An analytical development which can automatically and inexpensively extract the elastic moduli for three pavement layers from measured surface deflections.
4. An analytical method for deriving the frequency response spectrum of a pavement by using discrete, finite Fourier transforms of the measured input and output signals of an impulsive loading device. As an alternative to processing the digital data from input and output signals, use available analog equipment that can do the same thing in real time. Conceivably this could produce the same kind of information as the variable frequency device recommended under item 2, above.
5. A horizontal and vertical Dynaflect-type device for measuring horizontal and vertical displacement influence coefficients of the surface of a pavement. The development also requires the assembly of a finite element computer code for analyzing an overlay to be placed on the old pavement.

6. A stable reference line and associated measurement equipment for making long-term creep measurements of pavement surface deflections. At present, this device can best be used with measurement equipment like the Cox and Sons device and the Corps of Engineers van, both of which are capable of applying and holding a constant load on a pavement. Building this reference line equipment also entails the development of an analytical method for extracting viscoelastic properties of two layers.

7. An impulsive loading device that can be attached to the present Dynaflect and all of the electronic measuring equipment required to detect cracks by attenuation of the surface velocity between geophones.

All of these developments are within the scope of current technology and with sufficient funding can be expected to be developed in a reasonably short period of time. As they are developed, they are expected to substantially advance the fields of pavement evaluation, analysis, and design.

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APPENDIX A

SUMMARY OF STATE PAVEMENT RATING SYSTEMS

INTRODUCTION

There are three parts to Appendix A. The first part shows the questionnaire used in requesting information on the rating systems. The second part gives summaries of the various state pavement rating systems and the third part summarizes the rating systems for cities, counties and Canadian provinces. A standard format for summarizing the rating systems was adopted so that the important features of each could be readily compared. In some cases, a state's rating system was much more complex than could be explained sufficiently within the standard format. In such cases, the added complexity is recognized in the commentary. The letter accompanying the questionnaires was careful to point out that the information was being requested, not for the purpose of establishing standards, but for collecting the kind of information summarized in this appendix.

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TEXAS A&M UNIVERSITY
COLLEGE OF ENGINEERING
COLLEGE STATION TEXAS 77843

OFFICE OF THE DEAN

AREA CODE 713
TELEPHONE 845-6431

Dear

We are presently under contract with the Federal Highway Administration to determine the best methods of evaluating the structural and distress condition of a pavement. The purpose of the project is not to standardize methods but to determine what really needs to be measured and what instruments are best for measuring them.

Part of our Task A is to assess the visual rating methods used by the State Highway Departments on both rigid and flexible pavements to determine the relative weights each state has placed on each observable form of distress such as longitudinal cracking, transverse cracking, alligator cracking, rutting, patching, flushing (or bleeding) and so on. We would greatly appreciate receiving from you any reports which give the details of your present distress weighting system and how it is used in aiding decisions to do maintenance or rehabilitation work on the pavement.

In case you do not have such reports available, or it would be too difficult to assemble the information, we have included a form on which you could fill out the weights of each distress type. For example the Washington State Highway Department presently has a rating score range (deduct value) of 2 to 25 for alligator cracking. This rating score implies (as compared to other types of distress) that alligator cracking is the third most important form of distress following rutting and waves, sags and humps. Please return the completed form in the enclosed, self-addressed envelope.

Thank you very much for your assistance.

Yours very truly,

Robert L. Lytton
Associate Research Engineer

RLL:njs

Enclosures

FLEXIBLE PAVEMENT DISTRESS RATING CRITERIA

	Range of Rating Scores		Comments
	Minimum Rating	Maximum Rating	
Transverse Cracking			
Longitudinal Cracking			
Multiple Cracking (Beginning of alligator cracking)			
Alligator Cracking			
Rutting			
Raveling			
Patching			
Flushing (or bleeding)			
Waves, Humps			
Corrugations			
Other			

RIGID PAVEMENT DISTRESS RATING CRITERIA

	Range of Rating (or Deduct) Scores		Comments
	Minimum Rating	Maximum Rating	
ALL PAVEMENTS			
Surface Deterioration -Raveling -Scaling			
Spalling			
Longitudinal Cracking			
Patching			
Faulting			
Pumping			
Failures per Mile			
Blowups			
CONTINUOUSLY REINFORCED CONCRETE			
Crack Spacing			
% Intersecting Cracks			
JOINTED CONCRETE			
Spalled Joints			
Faulted Joints			
Cracked Panels			
Broken Panels			
Transverse Cracking			
Joint Spacing			
OTHER			

PAVEMENT RATING SYSTEM SUMMARIES

A standard format was used in summarizing the various rating systems reviewed. The format with explanatory remarks is as follows:

I. General

1. Basic Composition:

   - name of rating system or use descriptive terms
   - describe pavements for which the rating system is applicable (flexible and/or rigid)
   - describe source of inputs into rating system, e.g., visual observations and/or mechanical measurements

2. Measuring Equipment Used:

   - describe or name equipment used e.g., PCA Roadmeter, Mu Meter, etc.

3. Rating Team Composition:

   - describe number of rating teams and number of individuals in each team

4. Rating Frequency:

   - time between ratings

5. Other:

   - general information about the rating system, further explanatory comments, etc

II. Brief Outline of System

- outline major rating categories
- assign Maximum Available Points (if possible) to each major rating category, e.g., amount of points indicating the best possible pavement or Maximum Available Negative Points indicating the poorest pavement

- assign the point range (min/max) for the various distress or descriptive items in each major rating category
- describe formula used to arrive at a numerical rating (if possible) if other than a simple addition of the major rating categories.

III. <u>Summation</u>

- answer preselected questions and define overall numerical point range.

For those states which replied to our questionnaire, many either do not use a numerical type of rating system or did not send enough information to properly review. Therefore, a short statement of any known significant information was made without using the full format.

ALASKA

General: Alaska uses a sufficiency rating report based on a collective count of all surface failures per mile. The rating system applies only to flexible pavements. Alaska has no rigid pavements in their highway system.

ARIZONA

I. General

1. Basic Composition: A sufficiency rating system is used for both flexible and rigid pavement systems. Ratings are derived primarily from visual observations.

2. Measuring Equipment Used: Unknown

3. Rating Team Composition: Unknown

4. Rating Frequency: Every two years.

5. Other: Arizona is currently in the process of modifying existing rating system to include Mays Meter, Dynaflect and Mu Meter measurements. Additionally, features such as cracking, rutting, shoving will be measured on a 1,000 sq. ft. area at each mile post. Wear, weathering, popouts and other surface deficiencies will be rated on a scale from 1 to 5.

II. Brief Outline of System

1. Condition — Max Available Points 35

	Point Range Min	Point Range Max
A. Structural adequacy	0	17
B. Anticipated remaining life	0	13
C. Maintenance economy	0	5

2. Safety — Max Available Points 30

	Point Range Min	Point Range Max
A. Roadway width	0	8
B. Surface width	0	7

C.	Sight distance	0	10
D.	Consistency	0	5

3. Service — Max Available Points 35

		Point Range Min	Max
A.	Alignment	0	12
B.	Passing opportunity	0	8
C.	Surface width	0	5
D.	Ride quality	0	10

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

   <—— Better Pavements
   100 |——+——+——+——| 0
   Poorer Pavements ——>

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

ARKANSAS

General: Arkansas does not use a numerical weighting system for evaluating pavements at the present time. The Planning and Research Division has conducted inspection ratings for a few sections of pavement. These inspection ratings were conducted for both flexible and rigid pavements. The following are considered on each rating form:

1. Flexible Pavements

    A. Cracking
    B. Texture
    C. Patching
    D. Rut Depth

2. Rigid Pavements

    A. Corner cracking
    B. Edge cracking
    C. Longitudinal cracking
    D. Transverse cracking
    E. Scaling
    F. Spalling
    G. Faulting joints
    H. Concrete disintegration
    I. Pumping
    J. Loss of joint filling
    K. Slab settlement
    L. Slab heaving
    M. Patching
    N. Local reconstruction
    O. Surface roughness
    P. Surface drainage (ponding)
    Q. Shoulder condition
    R. General condition

CALIFORNIA

I. General

1. Basic Composition: California uses a condition rating number to rate flexible pavements and partially rates rigid pavements with a ride rating. The condition rating is derived from both mechanical and visual measurements.
2. Measuring Equipment Used: Cox Ride Meter.
3. Rating Team Composition: Five to six teams, two men each team.
4. Rating Frequency: Every two years.
5. Other: The rating of flexible pavements considers both a ride and defect score. Rigid pavements are currently rated by a ride score only.

II. Brief Outline of System

1. Ride Score — Max Available Negative Points 100

Score obtained from Cox Ride Meter with 100 points representing the roughest conditions.

2. Defect Score — Max Available Negative Points 310

	Point Range Min	Point Range Max
A. Alligatoring block cracks	0	96
B. Transverse cracks	0	24
C. Longitudinal cracks	0	48
D. Ravel	0	60
E. Rutting	0	48

F.	Patching	0	24
G.	Rainfall	0	10

3. Condition Rating = $\sqrt{\text{Ride Rating} \times \Sigma \text{ Defect Score}}$

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

   ← Better Pavements

   0 |——+——+——+——| 176

   Poorer Pavements →

3. Numerical rating adjusted for traffic? No, but traffic considered in maintenance decisions.

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

COLORADO

General: Colorado uses a sufficiency type of rating system. Unfortunately, part of the information package sent was not received precluding a complete examination of their system. It was determined from the information available that the rating includes measurements of skid and slope variance.

CONNECTICUT

General: Connecticut does not use a numerical weighting system for evaluating pavements. Visual inspections are made twice a year to determine priority lists of work requirements.

FLORIDA

I. General

1. Basic Composition: Florida uses a condition rating number to rate flexible pavements and will evaluate rigid pavements as soon as procedures are developed. The condition rating is derived from both mechanical and visual measurements.

2. Measuring Equipment Used: Mays Meter, six-foot straight edge and ruler.

3. Rating Team Composition: Five teams, two men each team

4. Rating Frequency: Annually

5. Other: The present system may be adjusted somewhat as data becomes available.

II. Brief Outline of System

1. Ride Rating — Max Available Points 100

Using Mays Meter, rating will be calculated from a value of 0 to 100 with 100 being the theoretically perfect ride.

2. Defect Rating — Max Available Points 100

Alloted initial value of 100 with defect values deducted from this number for various degrees of pavement distress. Defect values based on cracking, rutting and patching.

A. Cracking

% of Area (up to)	Deduct Points Nonconnected	Alligator	Spalling
25	5	7	10
50	10	15	20
75	15	22	30
100	20	30	40

Note: Can combine different types of cracking

B. Patching (per lane)

Deduct Points

5	Light (less than 50 ft^2/100 ft of lane)
10	Moderate (50 to 100 ft^2/100 ft of lane)
15	Severe (more than 100 ft^2/100 ft of lane)

C. Rutting

Avg. Depth	Deduct Points	Avg. Depth	Deduct Points
1/8"	5	5/8"	25
1/4"	10	3/4"	30
3/8"	15	7/8"	35
1/2"	20	1"	40

3. Final Rating = $\sqrt{\text{Ride Rating} \times \text{Defect Rating}}$

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

   ←——— Better Pavements
   100 |——|——|——|——| 0
   Poorer Pavements ———→

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

GEORGIA

I. General

1. Basic Composition: Georgia uses a numerical rating system for flexible pavements only. The rating system in use is mainly a function of mechanical measurements such as those made with the Dynaflect, Wisconsin Roadmeter and skid measurements. Additionally, a subjective visual rating is or can be performed.
2. Measuring Equipment Used: Dynaflect, Wisconsin Roadmeter and skid measurements.
3. Rating Team Composition: Unknown
4. Rating Frequency: Unknown
5. Other: The visual rating is based on a 0 to 10 scale with 10 representing very poor pavements and 0 very good or excellent pavements. This system is primarily used to evaluate the system described below.

II. Brief Outline of System

1. Serviceability Condition Max Available Negative Points 100

   Serviceability condition is determined by obtaining the average roughness per mile with the Wisconsin Roadmeter. A roughness count of 1200 is assigned a rating of 100 and a roughness count of 200 is assigned a rating of 0.

2. Structural Condition Max Available Negative Points 100

The Dynaflect is used in determining structural condition. The three deflection parameters calculated are Dynaflect Maximum Deflection, Surface Curvature Index and Base Curvature Index. The actual deflection parameters are compared against maximum deflection parameters which should exist at the present time in order to obtain a serviceability level of 2.5 at the end of the design period. This comparison results in an appropriate rating for the structural condition.

3. Skid Resistance Max Available Negative Points 100

Skid resistance is a function of the skid level at 40 mph and an adjustment factor which depends on mix type and vehicle speed. This information is placed into an equation to obtain the skid resistance rating.

4. Total Rating = Traffic Factor $\left[\dfrac{\text{Roughness} + 1.5\ \text{Skid} + 1.5\ \text{Structure}}{4}\right]$

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

$$\begin{array}{c} \longleftarrow \text{Better Pavements} \\ 0 \;|\!\!-\!\!-\!\!+\!\!-\!\!-\!\!+\!\!-\!\!-\!\!+\!\!-\!\!-\!\!|\; 100 \\ \text{Poorer Pavements} \longrightarrow \end{array}$$

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Unknown

HAWAII

General: Hawaii does not use a numerical rating system for evaluating pavements.

IDAHO

General: Idaho uses a sufficiency rating system similar to that used by Arizona. Idaho is currently reassessing its position relative to the present system.

INDIANA

I. General

1. Basic Composition: A sufficiency rating system is used to evaluate both flexible and rigid pavements. Ratings are obtained primarily from visual observations.

2. Measuring Equipment Used: None

3. Rating Team Composition: Two, two man teams from each district for each of the six districts in Indiana.

4. Rating Frequency: Annually

5. Other: Indiana uses separate rating systems for rural and urban highways. The two are similar with only slight variations The rural system is presented below.

II. Brief Outline of System

1. Geometrics — Max Available Points 60

		Point Range Min	Point Range Max
A.	Surface type	0	5
B.	Surface width	0	15
C.	Shoulder type	0	3
D.	Shoulder width	0	7
E.	Stopping sight distance	0	9
F.	Alignment consistency	0	12
G.	Passing opportunity	0	9

2. <u>Condition</u> Max Available Points 40

	Point Range Min	Max
A. Structural adequacy	0	22
B. Drainage adequacy	0	8
C. Rideability	0	5
D. Traffic control	0	5

III. <u>Summation</u>

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range.

   ← Better Pavements

   100 ├───┼───┼───┼───┤ 0

   Poorer Pavements →

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? No

KANSAS

I. General

1. Basic Composition: Kansas has a point rating system for both flexible and rigid pavements. Ratings are derived primarily from visual observations.

2. Measuring Equipment Used: Roughometer

3. Rating Team Composition: One man surveys entire state.

4. Rating Frequency: Unknown

5. Other: System based on a maximum of 100 points available for a theoretically perfect pavement. In the outline below is shown the maximum amount of points allowed for the various items listed.

II. Brief Outline of System

1. Flexible Pavements — Max Available Points 100

		Point Range Min	Point Range Max
A.	Surface information	0	4
B.	Transverse cracks	1	10
C.	Transverse crack type	1	3
D.	Longitudinal cracking	1	5
E.	Crack pouring	3	15
F.	Original roadway design	1	5
G.	Surface required	3	18
H.	Dilute seal	0	4
I.	Skid resistance	2	10
J.	Uniformity of surface texture and color	2	10

		Min	Max
K.	Wheel ruts	3	12
L.	Structural adequacy	1	4

2. Rigid Pavements — Max Available Points 100

		Point Range Min	Max
A.	Roughometer	1	4
B.	Curb Condition	1	4
C.	Joints Filled	0	10
D.	Undulations	1	10
E.	Scaling	1	4
F.	Faulting	1	4
G.	Spalling	1	4
H.	Structural adequacy	1	4
I.	Cracking		
	(1) Random	1	5
	(2) Longitudinal	1	6
	(3) Transverse	1	5
J.	Surface patching required	3	18
K.	"D" Cracking	2	8
L.	Skid resistance	1	4
M.	Crack pouring completed	0	10

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range.

<—— Better Pavements 16 or 18

100 |——+——+——+——| 18

Poorer Pavements ——>

3. Numerical rating adjusted for traffic? No

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

KENTUCKY

General: Kentucky uses a numerical rating system to establish priorities for their resurfacing program. The system applies to bituminous surfaces only and considers three categories. They are:

1. Service
2. Condition
3. Safety

LOUISIANA

I. General

1. Basic Composition: Louisiana uses a sufficiency rating for both flexible and rigid pavements. Ratings are derived from a combination of both visual and mechanical measurements.

2. Measuring Equipment Used: Roughometer (Mays Meter) and skid measurements.

3. Rating Team Composition: Unknown

4. Rating Frequency: Unknown

5. Other: Louisiana uses separate rating systems for rural and urban highways. Both systems are similar in composition. The rural rating system is presented below.

II. Brief Outline of System

1. Condition — Max Available Points 50

	Point Range Min	Point Range Max
A. Surface	5	20
B. Base and Subbase	1	10
C. Drainage	1	6
D. Subgrade	1	4
E. Roughometer	1	5
F. Remaining years of service	1	5

2. Service - volume/capacity ratio — Max Available Points 30

3. <u>Safety</u> Max Available Points 20

	Point Range Min	Point Range Max
A. Shoulder width	1	5
B. Surface width	1	5
C. Alignment	1	5
D. Skid number	1	5

III. <u>Summation</u>

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range.

   <—— Better Pavements

   100 |——+——+——+——| 14

   Poorer Pavements ——>

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

I. General

1. Basic Composition: Maine uses a numerical rating system employing weighting coefficients for flexible pavements. It is not known if Maine employs a separate system for rigid pavements. Ratings are derived from visual observations.

2. Measuring Equipment Used: None

3. Rating Team Composition: Unknown

4. Rating Frequency: Unknown

5. Other: Maine has more than one rating system some of which are mechanical and visual systems. Each system is designed for use on a specific problem or is directly correlated to the system outlined below. For the sytem outlined, ratings are obtained for the surfacing, base, and overall pavement structure. This is done to account for the distress as exhibited by their pavements.

II. Brief Outline of System

1. Characteristics	Weighting Coefficients Pavement	Base	Overall	Point Range Min	Max
A. Centerline Crack	0.08	0.02	0.05	1	5
B. Random Cracks	0.03	0.03	0.14	1	5
C. Hairchecks	0.05	0	0.02	1	5
D. Alligator Cracks	0.08	0.22	0.15	1	5
E. Temperature Cracks	0.17	0	0.08	1	5
F. Rutting	0.07	0.20	0.14	1	5
G. Distortion	0.05	0.23	0.14	1	5
H. Washboard	0.09	0.10	0.05	1	5
I. Pitting	0.29	0	0.14	1	5
J. General Overall	0.09	.09	0.09	1	5
	1.00	0.89	1.00		

2. Calculated Rating = Σ(characteristic point scores x weighting coeff.)

Note: All characteristic point scores are multiplied by the appropriate weighting coefficient and summed. This results in a pavement rating number between 1 and 5.

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

```
     <—— Better Pavements
  5 |——|——|——|——| 1
     Poorer Pavements ——>
```

3. Numerical rating adjusted for traffic? No

4. Is assistance of rating system used in determining maintenance priorities at this time? Unknown

MARYLAND

I. General

1. Basic Composition: Present system uses visual observations to determine pavement rating and is used to evaluate both flexible and rigid pavements.

2. Measuring Equipment Used: None

3. Rating Team Composition: One man assisted by local personnel.

4. Rating Frequency: Annually

5. Other: Maryland is currently participating in a research project to develop a Highway Serviceability Index. This index will consider surface roughness, visual ratings and skid resistance. The end result will be a method that assists in determining where safety and maintenance appropriations should be spent. Presented below is the current composition of the present rating system.

II. Brief Outline of System

1. Surface Rating — Max Available Points 40

Considers:
- A. Cracking
- B. Alligatoring
- C. Rutting
- D. Patching
- E. Raveling
- F. Defective joints
- G. Cracked panels
- H. Scaling

I. Cross section

J. Profile section

2. Drainage Rating — Max Available Points 25
3. Shoulder Rating — Max Available Points 10
4. Major Structures Rating — Max Available Points 10
5. Minor Structures Rating — Max Available Points 5
6. Roadside Rating — Max Available Points 5
7. Traffic Service Rating — Max Available Points 5

III. Summation

1. Can a resulting numerical rating be defined? Yes
2. If yes, define range:

   <—— Better Pavements

   100 |——|——|——|——| 0

   Poorer Pavements ——>
3. Numerical rating adjusted for traffic? No
4. Is assistance of rating system used in determining maintenance priorities at this time? Unknown

MICHIGAN

General: Michigan uses two procedures for determining resurfacing or reconstruction requirements. First, an annual sufficiency type of survey is performed which is a general opinion of the rater and involves no measurements. This survey is used to identify those roads which should be inspected in more detail. Secondly, a measurement of Present Serviceability Index (PSI) may be made as additional justification for resurfacing or reconstruction projects. For determining PSI, ride quality (using a General Motors type Travel Profilometer), linear feet of cracks, areas of patches, and rut depths (bituminous pavements only) are measured.

MINNESOTA

I. <u>General</u>

1. <u>Basic Composition</u>: Minnesota uses a condition rating system for both flexible and rigid pavements. Ratings are derived primarily from visual observations and from mechanical measurements.

2. <u>Measuring Equipment Used</u>: PCA Roadmeter, six foot long straight edge for measuring rut depth.

3. <u>Rating Team Composition</u>: One rating team for each of the nine districts, generally two men each team.

4. <u>Rating Frequency</u>: Annually

5. <u>Other</u>: Minnesota uses a Present Serviceability Rating (PSR) and Structural Rating (SR) combined to result in a Condition Rating (CR). Initially, they used three man teams to rate PSR but found this did not result in the desired accuracy. Therefore, the decision was made to use the PCA Roadmeter to achieve greatly improved results.

II. <u>Brief Outline of System</u>

Pavements are rated according to composition in three separate categories: bituminous, concrete and bituminous overlayed concrete pavements.

1. <u>Present Serviceability Rating</u> Max Available Points <u>5</u>

Determined for all three types of pavements with the PCA Roadmeter.

2. <u>Structural Rating</u> Max Available Points <u>5</u>

Bituminous Pavements

SR = Σ(% occurrence each deficiency x weighting factor), converted to a number between 0 and 5.

Deficiency	Weighting Factors
A. Transverse cracking	0.02
B. Longitudinal cracking	0.02
C. Multiple cracking	0.15
D. Alligator cracking	0.35
E. Rutting	0.15
F. Patching	0.30
	0.99

Concrete Pavements

SR = Σ(% occurrence each deficiency, converted to a number between 0 and 5 x weighting factor)

Deficiency	Weighting Factors
A. Joints	
(1) Spalled	0.25
(2) Faulted	0.10
B. Panels	
(1) Cracked	0.10
(2) Broken	0.10
(3) Faulted cracks	0.10
C. Patches	
(1) Area of 5 ft^2 or more	0.20
(2) Complete overlay	0.10
D. Scale	0.05
	1.00

Bituminous Overlaid Concrete Pavements

SR = Σ(% occurrence each deficiency, converted to a number between 0 and 5 x weighting factor)

Deficiency	Weighting Factors
A. Cracking	
(1) Slight transverse	0.05
(2) Severe transverse	0.30
(3) Slight longitudinal	0.05

(4) Severe longitudinal 0.30
(5) Multiple 0.10

B. Patching 0.20

1.00

3. Condition Rating $= \frac{PSR + SR}{2}$

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

$$\begin{array}{c} \longleftarrow \text{Better Pavements} \\ 5 \;|\!\!-\!\!-\!\!-\!\!+\!\!-\!\!-\!\!-\!\!+\!\!-\!\!-\!\!-\!\!+\!\!-\!\!-\!\!-\!\!|\; 0 \\ \text{Poorer Pavements} \longrightarrow \end{array}$$

3. Numerical rating adjusted for traffic? No, but Minnesota considers traffic in arriving at final maintenance priorities.

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

MISSISSIPPI

General: Mississippi does not use a rating system at the present time

MISSOURI

General: Missouri does not use written guidelines or rating criteria at this time.

MONTANA

General: Montana uses a sufficiency rating system. One individual rates all the highways maintained by the Department of Highways. A detailed summary of the Montana system was not received.

NEBRASKA

I. General

1. Basic Composition: A sufficiency rating system is used for both flexible and rigid pavements. Ratings are derived primarily from mechanical measurements with contributions from visual determinations.

2. Measuring Equipment Used: Dynaflect, Nebraska Roadmeter, skid measuring equipment conforming to ASTM E 274.

3. Rating Team Composition: Unknown

4. Rating Frequency: Every two years

5. Other: System as described below will be used for the 1975 rating survey and is a modified version of the one used for the 1973 survey.

II. Brief Outline of System

1. Condition — Max Available Points 40

	Point Range Min	Max
A. Structural Adequacy	0	20

(1) Flexible pavement: Computed from a formula which is a function of average daily traffic and dynaflect deflection.

(2) Rigid pavement: Two separate categories - one for rigid pavement with flexible overlay and one for rigid pavement without flexible overlay. A formula is used for both categories to compute structural adequacy. Formula for rigid pavement with flexible overlay is a

function of average daily traffic, thickness of concrete, thickness of asphaltic concrete overlay and roadmeter data. Formula for rigid pavement without flexible overlay is function of average daily traffic, thickness of concrete and roadmeter data.

	Point Range Min	Max
B. Roughness	0	20

(1) Flexible pavement: Computed from a formula which is a function of roadmeter data, cracking, patching and rut depth.

(2) Rigid pavement: Computed from a formula which is a function of roadmeter data, rigid pavement age and rigid pavement thickness for rigid pavements without flexible overlay. Function of roadmeter data and flexible overlay age for rigid pavements with flexible overlay.

2. Safety and Service — Max Available Points 60

	Point Range Min	Max
A. Surface width	0	10
B. Shoulder width and condition	0	10
C. Stopping sight distance	0	5
D. Passing opportunity	0	10
E. Consistency	0	7
F. Foreslopes	0	8
G. Skid	0	10

III. Summation

1. Can a resulting numerical rating be defined? Yes

<—— Better Pavements

2. If yes, define range: 100 |——+——+——+——| 0

Poorer Pavements ——>

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

NEW HAMPSHIRE

General: New Hampshire does not use a rating system for evaluating pavements at the present time. For flexible pavements, most of their observed pavement distress is either longitudinal or transverse cracking. New Hampshire has only a small amount of rigid pavement.

NEW JERSEY

General: New Jersey is currently in the process of establishing a sufficiency rating system for both flexible and rigid pavements. The rating system will be comprised of surface roughness as measured by the Mays Meter and physical measurement of pavement deterioration and failures. Three pavement categories will be rated: rigid, flexible, and composite (asphalt concrete overlays over portland cement concrete). Rigid pavements shall receive heavy weighting on spalling, cracking, and patching. Flexible pavement weighting shall concentrate on rutting, patching, and cracking. Composite pavements shall be primarily weighted on patching and degree of reflection cracking.

NEW MEXICO

I. General

1. Basic Composition: New Mexico uses a sufficiency type rating system for both flexible and rigid pavements. Ratings are derived from visual observations.

2. Measuring equipment Used: None

3. Rating Team Composition: One man

4. Rating Frequency: Annually

5. Other: Evaluation of New Mexico highways is not based solely on the point system as summarized below. If any highway section has a "critical deficiency" in any one of its major rating categories, the section is singled out for corrective action.

II. Brief Outline of System

1. Structural Adequacy — Max Available Points 50

	Point Range Min	Point Range Max
A. Foundation	0	10
B. Surface	0	30
C. Drainage	0	10

2. Safety — Max Available Points 20

3. Capacity — Max Available Points 30

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

← Better Pavements

100 |——|——|——|——| 0

Poorer Pavements →

3. Numerical rating adjusted for traffic? Yes

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

NORTH CAROLINA

General: For flexible pavements, North Carolina uses the rating system contained in HRB Digest #48, July, 1973, entitled "Surface Condition Rating System for Bituminous Pavements". North Carolina has no established rating system for rigid pavements at the present time, but the heaviest weights are placed on pumping, broken pavement, and intersecting cracks.

NORTH DAKOTA.

I. General

1. Basic Composition: North Dakota uses a pavement condition rating for flexible pavements only. This rating is derived form visual observations and used in conjunction with a Mays Meter survey and other considerations to establish maintenance priorities.

2. Measuring Equipment Used: None

3. Rating Team Composition: Unknown

4. Rating Frequency: Unknown

5. Other: A pavement with a score of 25 or greater is generally considered in need of immediate maintenance.

II. Brief Outline of System

1. Surface Cracking — Max Available Negative Points 24

	Point Range Min	Max
A. Transverse cracking	0	3
B. Fatigue cracking	0	3
C. Transverse crack widths	0	3
D. Longitudinal cracking	0	3
E. Crack spalling	0	4
F. Map cracking	0	3
G. Alligator cracking	0	5

2. Surface Distortion — Max Available Negative Points 13

	Point Range Min	Max
A. Rutting	0	5
B. Shoving	0	3

		Min	Max
C.	Potholes	0	3
D.	Raveling	0	2

3. Maintenance Effort — Max Available Negative Points 6

		Point Range Min	Point Range Max
A.	Scotch patching	0	2
B.	Mix patching	0	4

4. Other Considerations — Max Available Negative Points 6

		Point Range Min	Point Range Max
A.	Seal condition	0	3
B.	Shoulder condition	0	3

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range: 0 |——|——|——|——| 49
   <—— Better Pavements
   Poorer Pavements ——>

3. Numerical rating adjusted for traffic? Unknown

OHIO

General: Ohio does not use a statewide numerical rating system for pavement evaluation. Some of the field districts have developed various methods for surveying pavements with the main considerations being general maintenance costs and rideability.

OREGON

I. General

1. Basic Composition: Oregon uses a surface condition rating system for flexible pavements. Ratings are derived primarily from visual observations and a mechanical measurement.
2. Measuring Equipment Used: PCA Roadmeter
3. Rating Team Composition: Initially, five surface rating teams and one rideability team were used. Each team apparently consisted of two members. The present number of teams and team composition may be different.
4. Rating Frequency: Unknown
5. Other: Oregon presently does not incorporate a structural strength rating into their system such as that obtained with deflection measurements. They intend to do this provided a fast method of measuring surface deflections can be obtained.

II. Brief Outline of System

1. Surface Summation: The combined effects of alligatoring, patching, wheel rutting and traffic erosion.
2. Surface Rating: The combined effects of the surface summation and rideability. Rideability is measured with a PCA Roadmeter.
3. Surface Condition Rating: The combined effects of the surface rating and average daily traffic. This number (SCR) gives a rank order listing of the roads most in need of repair.

III. Summation

1. Can a resulting numerical rating be defined? Yes
2. If yes, define range: Not definable with information available
3. Numerical rating adjusted for traffic? Yes
4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

PENNSYLVANIA

General: Pennsylvania does not use a formal visual rating system for assessing pavement distress.

SOUTH CAROLINA

General: South Carolina does not use a formal procedure for evaluating pavements. Routine inspections by maintenance personnel are utilized to determine pavement maintenance requirements. During these inspections, the type and amount of cracking, rutting, patching, raveling and other distress types are considered.

SOUTH DAKOTA

General: South Dakota does not use a distress weighting system. Their correspondence does not state if they use some other type of rating system.

TENNESSEE

I. General

1. Basic Composition: Tennessee uses a pavement condition rating system based solely on visual observations. This one system is used to rate both flexible and rigid pavements.

2. Measuring Equipment Used: None

3. Rating Team Composition: One team rates the entire state. Number of members in this one team is unknown.

4. Rating Frequency: Annually

5. Other: None

II. Brief Outline of System — Max Available Points 25

1. Cross Section

Considers:
A. Uniformity of crown
B. Superelevation of curves
C. Raveling and/or spalling of pavement edges
D. Longitudinal depressions (rutting)

2. Profile — Max Available Points 25

Considers:
A. Transverse undulations (waves and corrugations)
B. Bumps
C. Dips
D. Riding quality

3. Surface Characteristics — Max Available Points 50

Considers:
A. Cracking
B. Potholes

C. Surface raveling and disintegration

D. Blow ups

E. Pumping

F. Bleeding

G. Patching

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

<——— Better Pavements
100 |———+———+———+———| 0
Poorer Pavements ———>

3. Numerical rating adjusted for traffic? No

4. Is assistance of rating system used in determining maintenance priorities at the time? Yes

TEXAS

I. General

1. Basic Composition: Texas uses a numerical rating system for both flexible and rigid pavements. Ratings are derived primarily from visual observations augmented by the Mays Meter to determine riding quality.
2. Measuring Equipment Used: Mays Meter
3. Rating Team Composition: Two men each team and one team for each of the 25 districts.
4. Rating Frequency: Annually
5. Other: The data collected for the system results in a computer printout stating a Pavement Rating Score, Shoulder Rating Score - Paved, Shoulder Rating Score - Unpaved, Roadside Rating Score, Drainage Rating Score and Traffic Services Rating Score.

II. Brief Outline of System

1. Pavement Rating Score — Max Available Points 100

PRS = $100 - \Sigma$ deduct points for distress

Type of Distress for Flexible Pavement	Point Deduct Range Min	Point Deduct Range Max
A. Rutting	0	15
B. Raveling	0	20
C. Flushing	0	20
D. Corrugations	0	20
E. Alligator cracking	0	25
F. Patching	0	20

G. Longitudinal cracking	0	25
H. Transverse cracking	0	20
I. Failures/mile	0	40
J. Mays Meter	0	50

Type of Distress for Rigid Pavement	Point Deduct Range Min	Max
A. Pumping	0	60
B. Failures/mile	0	40
C. Surface deterioration	0	60
D. Spalling	0	60
E. Longitudinal cracking	0	25
F. Patching	0	20
G. Faulting	0	40
H. Crack spacing (CRCP only)	0	40
I. % intersecting cracks (CRCP only)	0	40
J. Transverse cracking		
(a) joint spacing < 20 feet (Jointed only)	0	40
(b) joint spacing > 20 feet (Jointed only)	0	30
K. Joints (joint sealer condition) (Jointed only)	0	20
L. Mays Meter	0	50

2. Shoulder Rating Score - Paved　　　Max Available Points 100

$$SRS = 100-1.428\ \Sigma\ (\text{deduct points})$$

Deduct Points Function of:

A. Ride

B. Contrast

C. Pavement edge

D. Shoulder edge

E. Cracks

F. Raveling

G. Vegetation

3. Shoulder Rating Score - Unpaved — Max Available Points 100

$$SRS = 100-5.00\ \Sigma\ (\text{deduct points})$$

Deduct Points Function of:

A. Pavement edge

B. Rutting, corrugations, loose rock

4. Roadside Rating Score — Max Available Points 100

$$RRS = 100-2.5\ \Sigma\ (\text{deduct points})$$

Deduct Points Function of:

A. Litter

B. Mowing

C. Vegetation

D. Slope erosion

5. Drainage Rating Score — Max Available Points 100

$$DRS = 100-3.333\ \Sigma\ (\text{deduct points})$$

Deduct Points Function of:

A. Culverts

B. Ditches, outfall, channels

C. Roadside drainage

6. Traffic Services Rating Score — Max Available Score 100

$$TRS = 100-2.0\ \Sigma\ (\text{deduct points})$$

Deduct Points Function of:

A. Guardrails

B. Signs

C. Delineators

D. Striping

e. Auxiliary markings

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:

   <—— Better Pavements

   100 |——+——+——+——| 0

   Poorer Pavements ——>

3. Numerical rating adjusted for traffic? No

4. Is assistance of rating system used in determining maintenance priorities at this time? No, but it will be in the future. System is in the process of being implemented.

UTAH

I. General

1. Basic Composition: A Pavement Evaluation System is used for flexible pavements. It is not known if a separate system is used for rigid pavements. The evaluation is derived primarily from mechanical measurements and from visual determinations.

2. Measuring Equipment Used: Dynaflect, Cox Roadmeter and Mu Meter.

3. Rating Team Composition: Two teams, two men each team.

4. Rating Frequency: New pavements evaluated every second or third year. Older pavements are evaluated annually.

5. Other:

A. The system as described is part of an overall pavement management system which is being developed. The PMS will be a management tool and store the following types of data:

(1) Geometrics - widths, grades, etc.

(2) Pavement design

(3) Construction control

(4) Environmental conditions

(5) Maintenance activities

(6) Pavement rehabilitation

(7) Traffic data

(8) Pavement evaluation

B. The Pavement Evaluation System does not as yet compute an overall numerical pavement rating. Certain distress items in the system are assigned numerical ratings from 1 (very poor) to 5 (excellent).

II. <u>Brief Outline of System</u>: The following items listed under each subheading are collected, computed (if required) and displayed on a computer print-out.

1. <u>Structural</u>

   A. Deflection readings for each of 5 sensors at each test site

   B. Average Dynaflect Maximum Deflection (DMD)

   C. Surface Curvature Index (SCI)

   D. Base Curvature Index (BCI)

   E. Predicted remaining structural life in 18 kip axle loads and years.

   F. Bituminous overlay thickness required for pavement to achieve 10 more years of structural life from the time the measurements were taken

   G. Condition statement based on DMD, SCI and BCI.

2. <u>Serviceability</u>

	Point Range Min	Point Range Max
A. Present serviceability index	1	5
B. Predicted remaining serviceability life in 18 kip axle loads and years until the pavement reaches the terminal serviceability index.		

3. <u>Slipperiness</u>

   A. Skid Index valves from the Mu Meter

   B. Predicted remaining safe skid resistance life in traffic loads and years

4. Surface Defects

A. Transverse cracking (L.F. per 1000 ft^2)

B. Longitudinal cracking (L.F. per 1000 ft^2)

C. Load associated cracking (ft^2 per 1000 ft^2)

D. Patching (ft^2 per 1000 ft^2)

E. Average condition of transverse and longitudinal cracks:

	Point Range Min	Point Range Max
(1) Opening	1	5
(2) Abrasion or erosion	1	5
(3) Multiplicity	1	5
F. Average surface wear	1	5
G. Average weathering	1	5
H. Average pop outs	1	5
I. Average uniformity	1	5

J. Average rut depth (in.)

III. Summation

1. Can a resulting numerical rating be defined? No

2. If yes, define range: N/A |—|—|—|—| N/A

<—— Better Pavements
Poorer Pavements ——>

3. Numerical rating adjusted for traffic? The evaluation system does consider traffic.

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

VERMONT

General: Vermont uses a flexible pavement condition survey to monitor the results of a transverse cracking study. The survey is a function of rideability, objective and subjective data. Rideability is an estimate of riding quality ranging from 0 (poor) to 5 (excellent). The objective data consists of measurements of transverse cracking, longitudinal load cracking, alligator cracking and rutting. Subjective data considers pitting, raveling, texture, settlement, bleeding and loss in matrix. This method does not result in an overall numerical rating.

VIRGINIA

General: Virginia uses a rating system for both flexible and rigid pavements. The percentage of weighting given a specific distress factor is as follows:

1. Flexible Pavements	% of Total Rating Min	Max
A. Transverse cracking	10	-
B. Longitudinal cracking	10	-
C. Multiple cracking	10	-
D. Alligator cracking	5	-
E. Rutting	5	-
F. Flushing	5	-
G. Waves, Humps	2	-
H. Raveling	1	-

2. Rigid Pavements - All Types	% of Total Rating Min	Max
A. Patching	30	-
B. Pumping	20	-
C. Spalling	20	-
D. Faulting	10	-
E. Scaling	5	-
F. Longitudinal cracking	1	-
Jointed		
A. Spalled joints	25	-
B. Faulted joints	10	-
C. Transverse cracking	5	-
D. Cracked panels	1	-
E. Broken panels	1	-
Continuously Reinforced		
A. % Intersecting cracks	1	5

I. General

1. Basic Composition: Washington uses a numerical rating system for both flexible and rigid pavements. Ratings are derived from both visual observations and mechanical measurements.

2. Measuring Equipment Used: PCA Roadmeter

3. Rating Team Composition: Four, two man teams

4. Rating Frequency: Every two years

5. Other: The rating system has recently been changed from using subjective ride ratings to ride measurements as determined by the PCA Roadmeter. Certain categories of defect ratings will also be changed or rearranged. A summary outline of the previously used system is presented below.

II. Brief Outline of System

1. Ride Rating — Max Available Points 100

RR = 100-(10 x Deduct Points)

	Point Deduct Range Min	Max
A. Ride Score	0	10

2. Defect Rating — Max Available Points 100

DR = 100-Σ Deduct Points

Flexible Pavement

	Point Deduct Range Min	Max
A. Rutting	0	40
B. Waves, sags, humps	0	40

		Min	Max
C.	Alligator cracking	0	25
D.	Corrugations, potholes	0	20
E.	Longitudinal cracking	0	25
F.	Transverse cracking	0	15
G.	Patching	0	15

<u>Rigid Pavement</u>

		Point Deduct Range Min	Max
A.	Cracking	0	50
B.	Raveling, disintegration, pop outs, scaling	0	50
C.	Spalling at joints and cracks	0	50
D.	Pumping, blowing	0	45
E.	Blowups	0	15
F.	Faulting, curling, warping, settlement	0	30
G.	Patching	0	15

Note: Total of deduct points for both types of pavement cannot exceed 100 points.

3. <u>Final Rating</u> = $\sqrt{\text{Ride Rating} \times \text{Defect Rating}}$

III. <u>Summation</u>

1. Can a resulting numerical rating be defined? <u>Yes</u>

2. If yes, define range:

   <—— Better Pavements
   100 |——+——+——+——| 0
   Poorer Pavements ——>

3. Numerical rating adjusted for traffic? <u>No</u>

4. Is assistance of rating system used in determining maintenance priorities at this time? <u>Yes</u>

WEST VIRGINIA

General: West Virginia does not use a standard pavement evaluation method at the present time. Provided was a rank order listing of what they consider the most important distress variables for both flexible and rigid pavements. The following flexible pavement variables are listed by order of importance: patching, rutting, waves, humps, corrugation, skid resistance, flushing, alligator cracking, and longitudinal cracking. For jointed rigid pavements, the following were listed by order of importance: faulted joints, joint spacing, broken panels, spalled joints, transverse cracking, and cracked panels.

WISCONSIN

General: Wisconsin uses a present serviceability index evaluation procedure. Lineal feet of cracking and rut depth measurements are obtained for use in the PSI formulas.

ONTARIO

General: Ontario uses a surface condition rating system based on determinations of ride quality and distress variables. The rating system as described is for flexible pavements. New rating techniques are being developed for rigid pavements. Ride quality is generally determined by experienced personnel which make a subjective assessment of pavement roughness. Ride ratings can range from "excellent" to "very poor". Distress variables are arranged into three groups and are identified as follows:

1. Surface Defects
    - A. Loss of aggregate
    - B. Raveling
    - C. Flushing
2. Surface distortion or deformation
    - A. Rippling
    - B. Shoving
    - C. Rutting
    - D. Distortion
3. Cracking
    - A. Longitudinal wheel track cracking
    - B. Longitudinal mid-lane crack
    - C. Centerline crack
    - D. Meandering crack
    - E. Pavement edge crack
    - F. Transverse crack
    - G. Random crack
    - H. Alligator crack

I. Settlement crack

J. Miscellaneous

Each item of distress is described by a standardized word description which considers the severity and extent of each type of distress. Using these standard descriptions and the attached sheet reproduced from Ontario's "Manual for Condition Rating of Flexible Pavements - Distress Manifestations", the selection of the condition rating score can be made.

A Guide for the Estimation of Pavement Condition Rating and Priority for Flexible Pavements		
Reconstruct within 2 years	0 - 20	Pavement is in poor to very poor condition with extensive severe cracking, alligatoring and dishing. Ridability is poor and the surface is very rough and uneven.
Reconstruct in 2 - 3 years	20 - 30	Pavement is in poor condition with moderate alligatoring and extensive severe cracking and dishing. Ridability is poor and the surface is very rough and uneven.
Reconstruct in 3 - 4 years	30 - 40	Pavement is in poor to fair condition with frequent moderate alligatoring and extensive moderate cracking and dishing. Ridability is poor to fair and surface is moderately rough and uneven.
Reconstruct in 4 - 5 years or resurface within 2 years with extensive padding	40 - 50	Pavement is in poor to fair condition with frequent moderate cracking and dishing, and intermittent moderate alligatoring. Ridability is poor to fair and surface is moderately rough and uneven.
Resurface within 3 years	50 - 65	Pavement is in fair condition with intermittent moderate and frequent slight cracking, and with intermittent slight or moderate alligatoring and dishing. Ridability is fair and surface is slightly rough and uneven.
Resurface in 3 - 5 years	65 - 75	Pavement is in fairly good condition with frequent slight cracking, slight or very slight dishing and a few areas of slight alligatoring. Ridability is fairly good with intermittent rough and uneven sections.
Normal maintenance only	75 - 90	Pavement is in good condition with frequent very slight or slight cracking. Ridability is good with a few slightly rough and uneven sections.
No maintenance required	90 - 100	Pavement is in excellent condition with few cracks. Ridability is excellent with few areas of slight distortion.

KING COUNTY, WASHINGTON

I. General

1. Basic Composition: King County has under consideration a Present Maintenance Rating (PMR) for flexible pavements. Ratings are derived primarily from visual observations augmented by the Cox Roadmeter to determine riding quality.

2. Measuring Equipment Used: Cox Roadmeter. Additional equipment to be used if warranted is a skid trailer conforming to ASTM E 274 and/or a Benkelman Beam. Other types of deflection measuring devices may be used in the future.

3. Rating Team Composition: Unknown

4. Rating Frequency: Annually

5. Other:

   A. A rating given a pavement not only assigns numerical values to the various distress descriptions but also lists the type of maintenance required for each kind of distress and when the maintenance should be performed.

   B. Rating system results in a rank order listing of all pavements with those in the poorest condition having the lowest numerical value.

   C. Deflection and skid measurements for a given pavement are conducted after the PMR has been obtained and indicates the need for such measurements.

   D. Rating system also considers roadside hazards and shoulder distress.

II. Brief Outline of System

1. Paving Distress — Max Available Points 60

		Point Range Min	Point Range Max
A.	Corrugations, shoving, slippage	0	10
B.	Flushing	0	5
C.	Raveling	0	10
D.	Rutting	0	10
E.	Alligator cracking	0	10
F.	Longitudinal cracking	0	5
G.	Transverse cracking	0	5
H.	Waves, sages, humps	0	5

Note: Each item listed above is a function of % area affected and severity.

2. Roughness — Max Available Points 100 or 125

Roughness count made with Cox Roadmeter converted to Present Serviceability Rating (PSR) on a scale from 0 to 5 with 5 being the best possible ride.

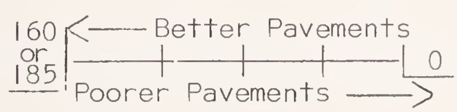
Then: Roughness = PSR x 20 for major and secondary roads or 25 for collector and access roads

3. PMR = Roughness + Σ Paving Distress Points

III. Summation

1. Can a resulting numerical rating be defined? Yes

2. If yes, define range:



3. Numerical rating adjusted for traffic? No, but the overall maintenance method does.

4. Is assistance of rating system used in determining maintenance priorities at this time? Yes

CORPUS CHRISTI, TEXAS

General: The city of Corpus Christi assigns a subjective numerical value ranging from 100 to 60 for each pavement section. The value of 100 represents a pavement in perfect condition whereas 60 represents a pavement that has completely deteriorated. The pavement sections are surveyed annually and the inputs assist in development of the annual maintenance program. Additionally, ditches and shoulders are rated.

WACO, TEXAS

General: The City of Waco uses a maintenance/priority system to assist in determining maintenance budgeting. Visual inspections of the pavement are made annually. These inspections determine the five types of maintenance functions required for a pavement section with the functions defined as: surface overlay, seal coat, mix seal, crack/joint maintenance and reconstruction. After the required maintenance function has been selected, priorities are chosen for the section within that maintenance function. Priority selections are based on such items as % of area exhibiting base failures, surface failures, loss of aggregate, etc.

APPENDIX B

CURRENT NON-DESTRUCTIVE EVALUATION OF PAVEMENT STRUCTURES

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APPENDIX B

CURRENT NON-DESTRUCTIVE EVALUATION OF PAVEMENT STRUCTURES

This Appendix contains a description of the current non-destructive testing techniques being used for the structural evaluation of pavements. It is divided into the following four major sections which categorize the various types of testing techniques:

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

Each major section contains a discussion of the general principles involved in testing as well as a description of specific equipment that is in use.

I. Static Deflections

General

Measurement systems that determine the pavement response to slowly applied loads are generally termed static deflection systems to distinguish them from various dynamic measurement systems now in use or under development. In static measurement systems, loads are applied by slowly driving to or away from a measurement point with a load wheel, or, they can be applied by reacting against a stationary truck frame. Unquestionably the most serious problem with these measurement devices is the difficulty of obtaining a suitable immovable reference for making deflection measurements. Because of this problem, question can be raised concerning the absolute accuracy of these measurement systems. Never-the-less, so much pavement performance experience has been related directly to them that they have become extremely important in the art of structural pavement evaluation (1).

Testing Equipment

Plate Bearing: This test has been used extensively to evaluate the modulus of subgrade reaction, "K", for use in concrete pavement design based upon Westerguard's analysis technique. It has also been employed in several flexible pavement design procedures. The Navy Method is an example (2). The test has also been used as a non-destructive pavement evaluation technique by some highway agencies.

Basically the test as used for pavement evaluation, consists of loading a circular plate on the surface of a pavement with a hydraulic jack that reacts against a truck frame (See Figure B.1). The deflection of the plate is measured with a dial gauge fastened to a cantilever beam that is supported on a stand placed as far away from the plate as possible. The load on the plate is increased until a specified deflection is reached.

This test is quite simple but time consuming. It requires about 30 minutes for a single test.

Curvature Meter: This device was developed to estimate the radius of curvature of the deflection basin produced by a wheel load. One version of the device is shown in Figure B.2. Basically it consists of a beam which is supported on the pavement by fixed probes at the ends of the

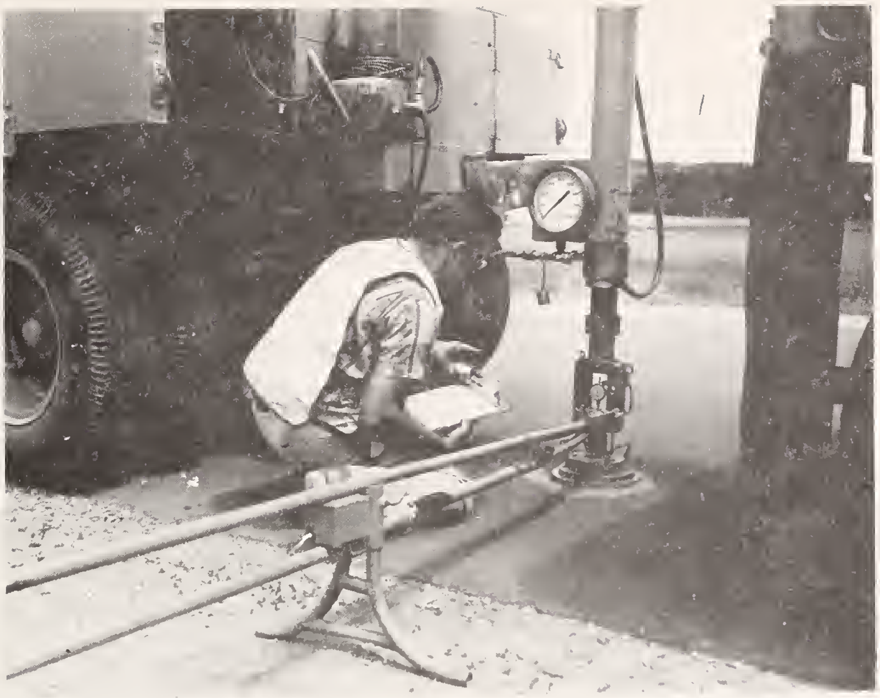


Figure B.1 Plate bearing test

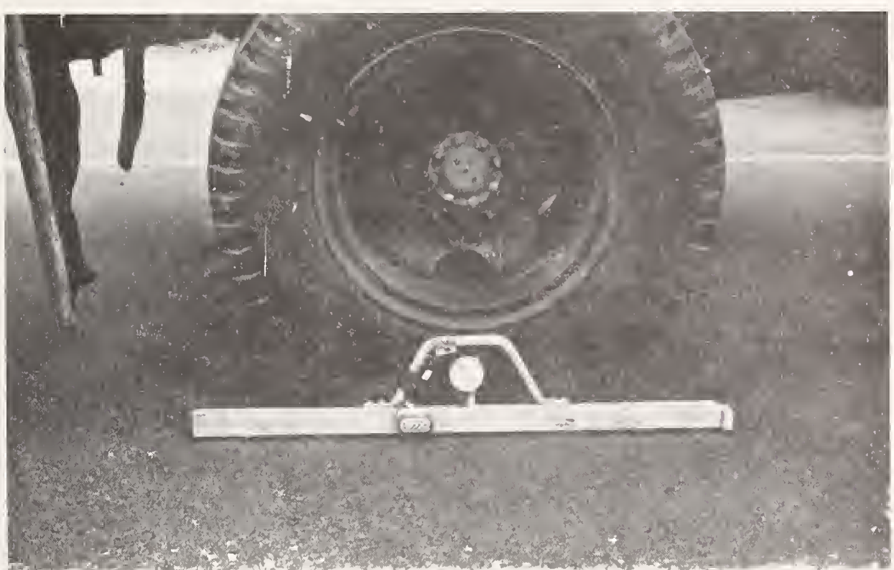


Figure B.2 Curvature meter

beam. It has a spring loaded probe in the center of the beam which rests on the pavement. The change in the position of this probe caused by moving a load wheel to or away from the beam is used to calculate the radius of curvature of the deflection basin.

This device is inexpensive and very simple to operate. One individual curvature measurement can be made in approximately 60 seconds and as many as 300 such measurements can be made in a working day.

Benkelman Beam: This instrument was developed and used at the WASHO Road Test to measure deflections on test sections of flexible pavements (3). Analysis of this road test data indicated that Benkelman Beam deflections could be used as an indicator of pavement performance. The instrument, shown in Figure B.3, is probably the most widely used device in the world today for the measurement of pavement deflections.

The Benkelman Beam is an 8-foot probe which is cantilevered from a reference beam that rests on the pavement. Deflections are determined by measuring the amount the probe pivots with respect to the reference beam. For measurements it is common practice to place the probe between the rear dual tires of a loaded single axle truck. As the truck is slowly driven away the maximum rebound deflection is determined. Generally the reference beam supports are considered to be far enough away from the load wheel to be outside of the influence of the deflection basin. However, because this assumption is often very inaccurate, particularly in stiffer pavements, some procedures have been developed for making corrections to measurements to account for motion of the reference beam supports (4).

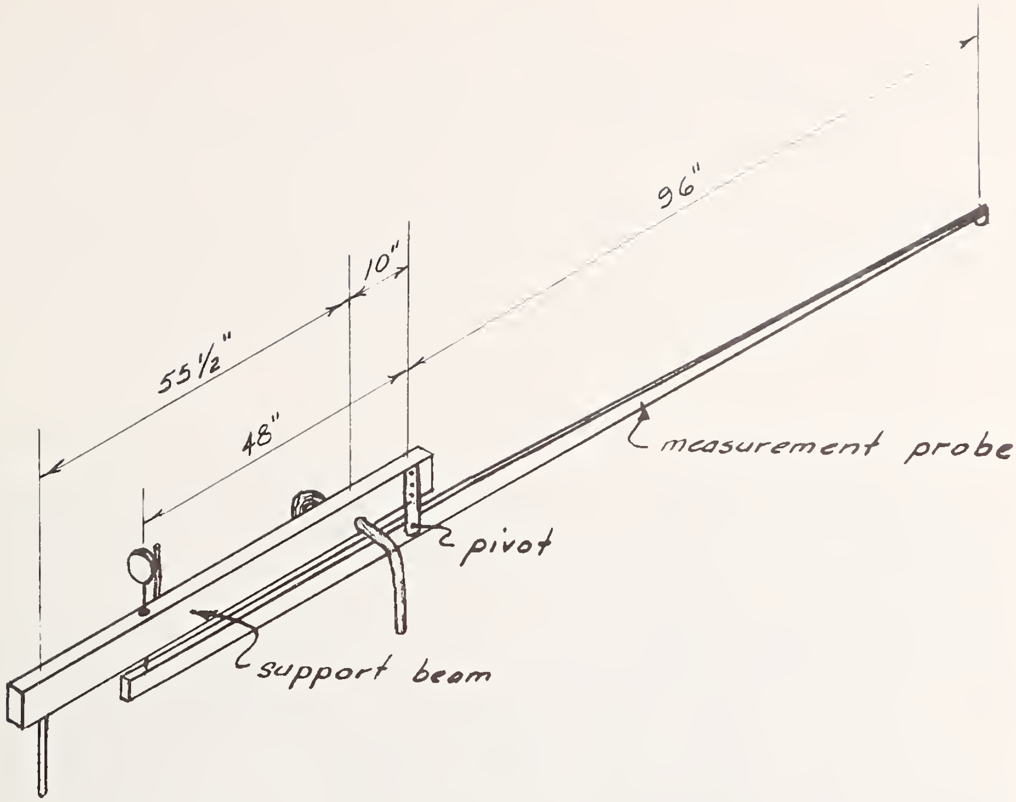


Figure B.3 Benkelman beam

The device is inexpensive and very simple to operate. One individual deflection measurement can be made in approximately 60 seconds and as many as 300 can be made in a normal working day.

Traveling Deflectometer: This instrument was developed by the California Division of Highways. It is basically an automated adaptation of the Benkelman Beam.

The device, shown in Figure B.4, is a truck trailer unit that is loaded to 18,000 pounds on the rear single axle. Two Benkelman Beam probes are mounted on a movable frame that is attached to the trailer. As the truck is in motion the probe frame is moved to its most forward position and then detached so that it rests on the pavement while the truck continues its forward motion. Simultaneous deflections for both rear wheels are determined when the rear axle reaches the probes. The frame is then moved to its most forward position and the cycle is repeated. It continuously measures and records deflections at twenty foot intervals as the truck travels at a speed of about one half mile per hour. In a working day the instrument can record about 1000 pairs of deflection measurements which represents a survey of nearly four miles.

LaCroix Deflectograph: This instrument is very similar to the California Traveling Deflectometer. It too is basically an automated adaptation of the Benkelman Beam; however, the beams are shorter. Thus, the support frame which is used as the reference base for measurements is more greatly influenced by the deflection basin. The instrument is available commercially from the Laboratoire Central Des Ponts Et Chaussees, 58 Boulevard Lefebvre,75732 Paris Cedex 15.

Figure B.4 California Traveling Deflectometer

The device, shown in Figure B.5, is a truck unit which is loaded to 14,000 pounds on its rear single axle. Two measurement probes are mounted on a movable frame. As the truck is in motion the probe frame is moved to its most forward position and released while the truck continues its forward motion. When the rear axle reaches the probes the deflections are determined and plotted on a strip chart recorder. Then the next cycle begins. The probes are shorter and the probe frame is smaller than the deflectometer enabling a complete cycle to be accomplished in 11 feet. The unit travels at a speed of about 1.2 mph and records deflections every 11 feet. In a working day the device can measure and record more than 4000 pairs of measurements which represents a survey of about 9 miles. Although this device has been used extensively in Europe, little or no use of it has been made in the United States.

Application

In general, highway engineers agree that deflections taken during the same season of the year provide an indication of the future useful life that can be expected for a pavement. That is, pavements that have high deflections will experience more rapid deterioration than similar pavements subjected to similar environmental conditions that have low deflections. In addition, the allowable deflection for a given pavement to have a specific anticipated life is related to the type of pavement structure. For example a gravel flexible base pavement can withstand higher deflections than a similar pavement structure having a soil cement base. Based upon these factors many evaluation procedures exist for determining the anticipated useful life (or the load capacity) of pavements based upon static deflections. In addition, several overlay

Figure B.5 LaCroix Deflectograph (upper) and close up of measurement probes (lower) design procedures are based upon measured static deflections. Valuable experience and judgment are embedded in these evaluation and design methods and any design procedure that is proposed in the future should be able to show a substantial dependence upon or correlation with these procedures.

The major deficiency in these evaluation techniques is their dependence upon some type of reference frame for deflection basin measurements. Such a reference frame is influenced to various degrees by the characteristics of the deflection basin itself. As a result the measurements are somewhat inaccurate and it is difficult to relate them to material properties of the pavement for extrapolation of past design experience to new and untried materials.

2. 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. Basically they all induce a steady state sinusoidal vibration in the pavement with a dynamic force generator. The dynamic force is superimposed upon the static force exerted by the weight of the force generator (See Figure B.6). The magnitude of the peak-to-peak dynamic force is less than twice the static force so that the vibrator continually applies a compressive load (of varying magnitude) to the pavement; thus, the vibrator never bounces off the pavement. The characteristics of the pavement are determined by measuring the deflection produced by the

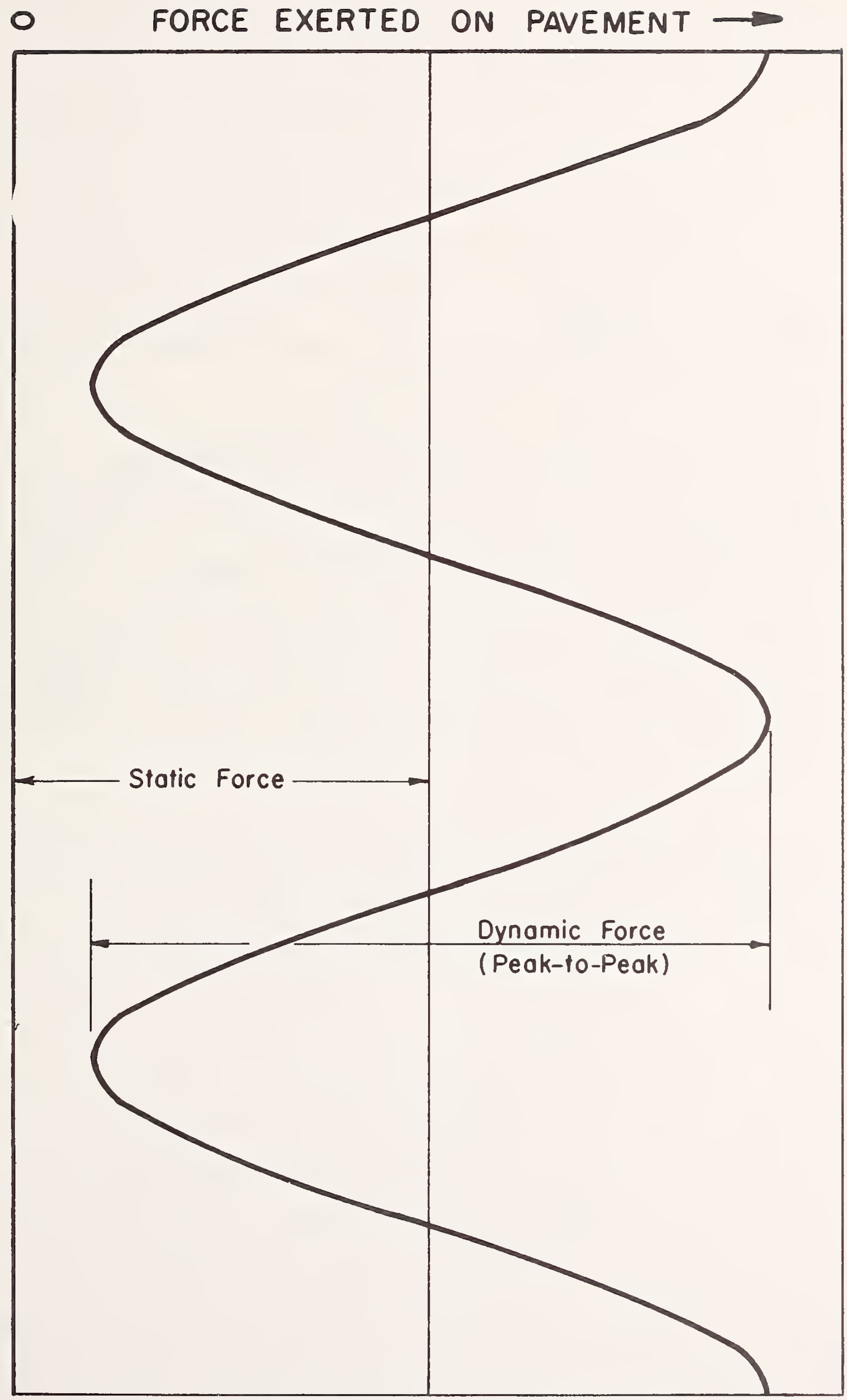


Figure B.5 Typical output of dynamic force generator dynamic force. When one considers the difficulty in obtaining a reference point for deflection measurements, the real advantage of a steady state dynamic deflection measurement system becomes apparent. An inertial reference can be employed to measure 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 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 good velocity sensor; for example, Mark Products 2Hz L-1H Geophone, are shown in Figure B.7. 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 2Hz 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,

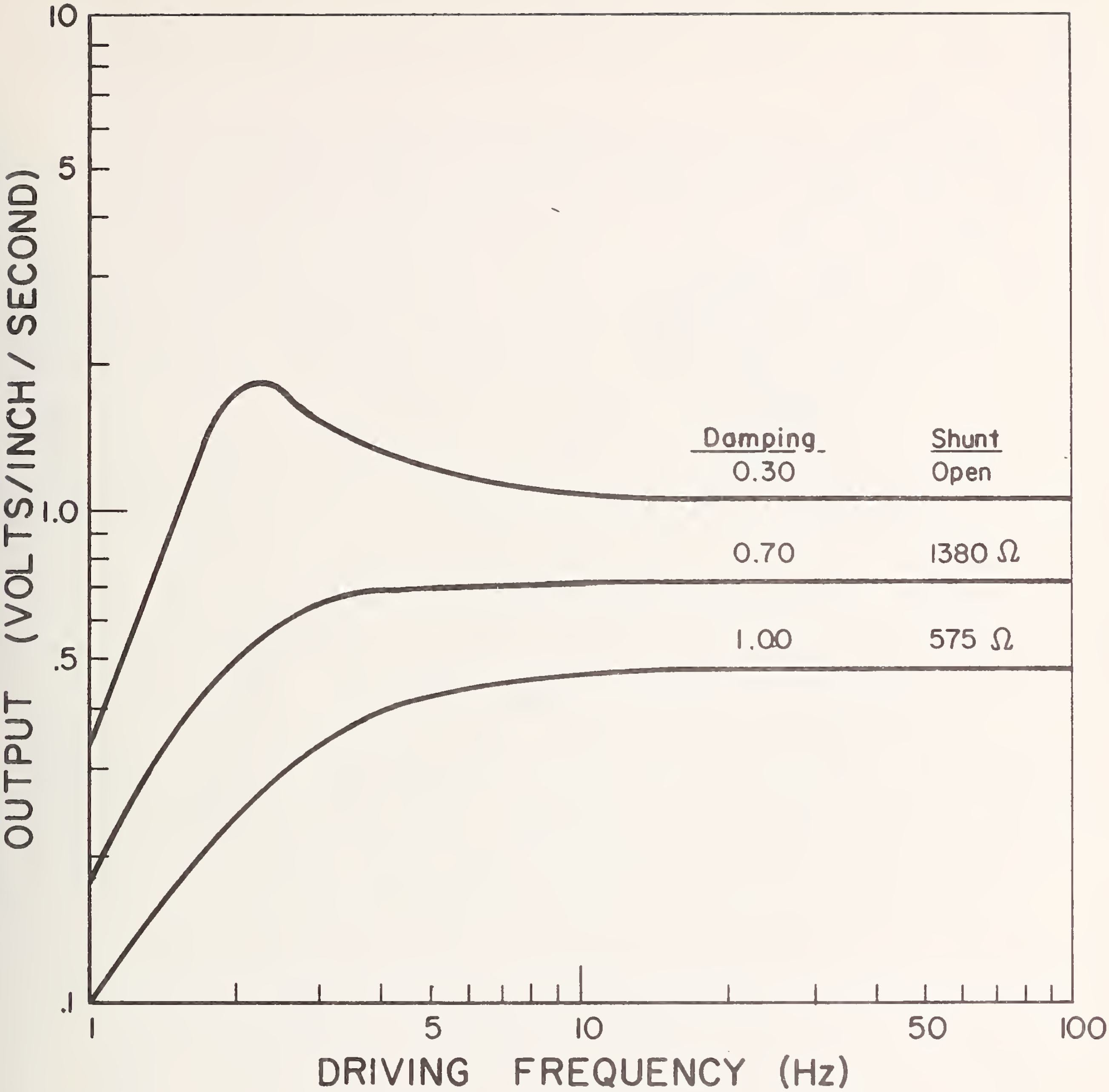


Figure B.7 Velocity response characteristics of a 500Ω, 2Hz geophone represents the velocity for any frequency above about 15 Hz.

Figure B.8 shows the output per unit of displacement for the same sensor illustrated in Figure B.7. 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 Figure B.9. 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. Figure B.10 gives the displacement response of the same sensor shown in Figures B.7 and B.8 when its output is integrated with an integrator having characteristics as shown in Figure B.9. 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

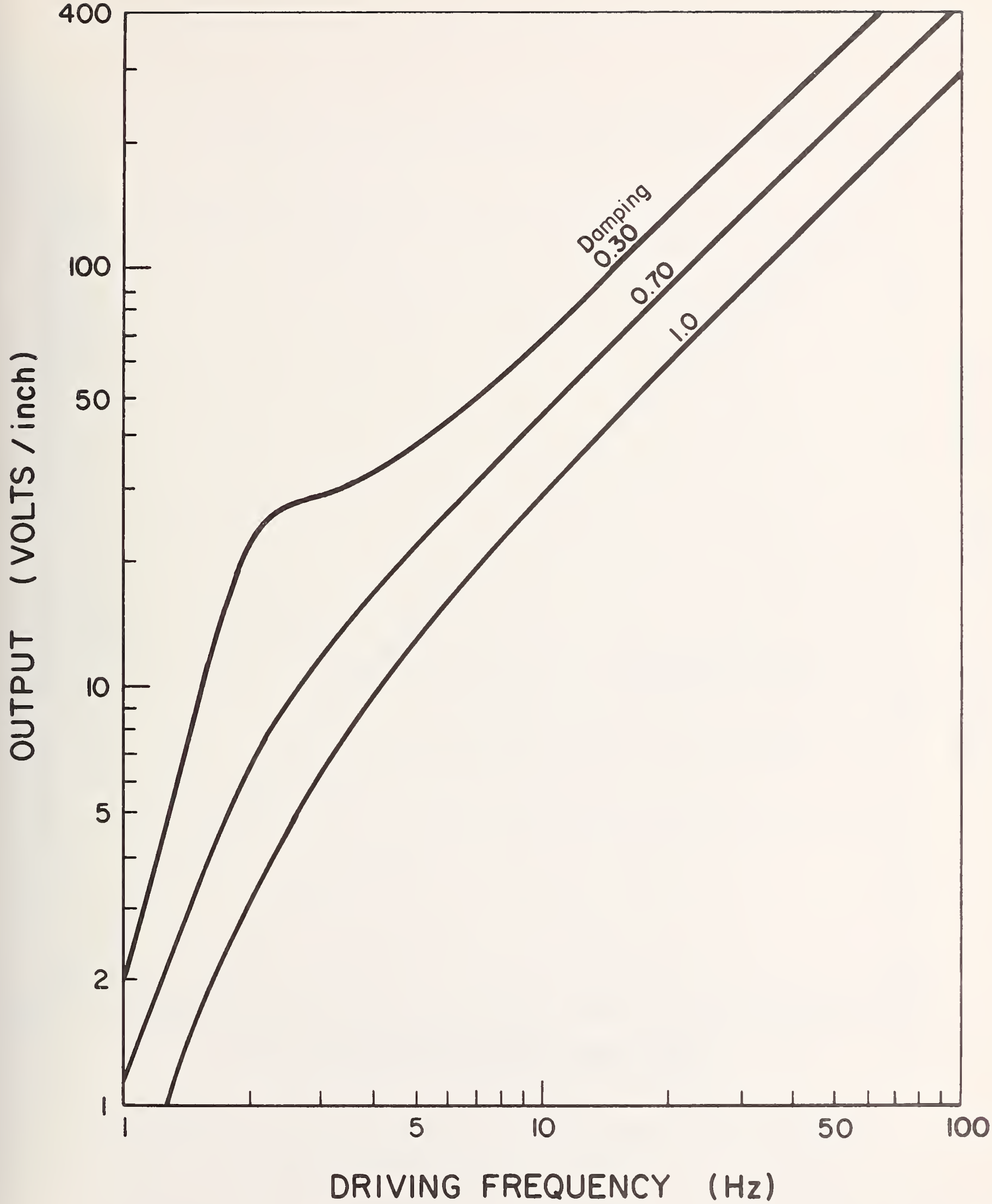


Figure B.8 Displacement response characteristics of a 500Ω, 2 Hz geophone

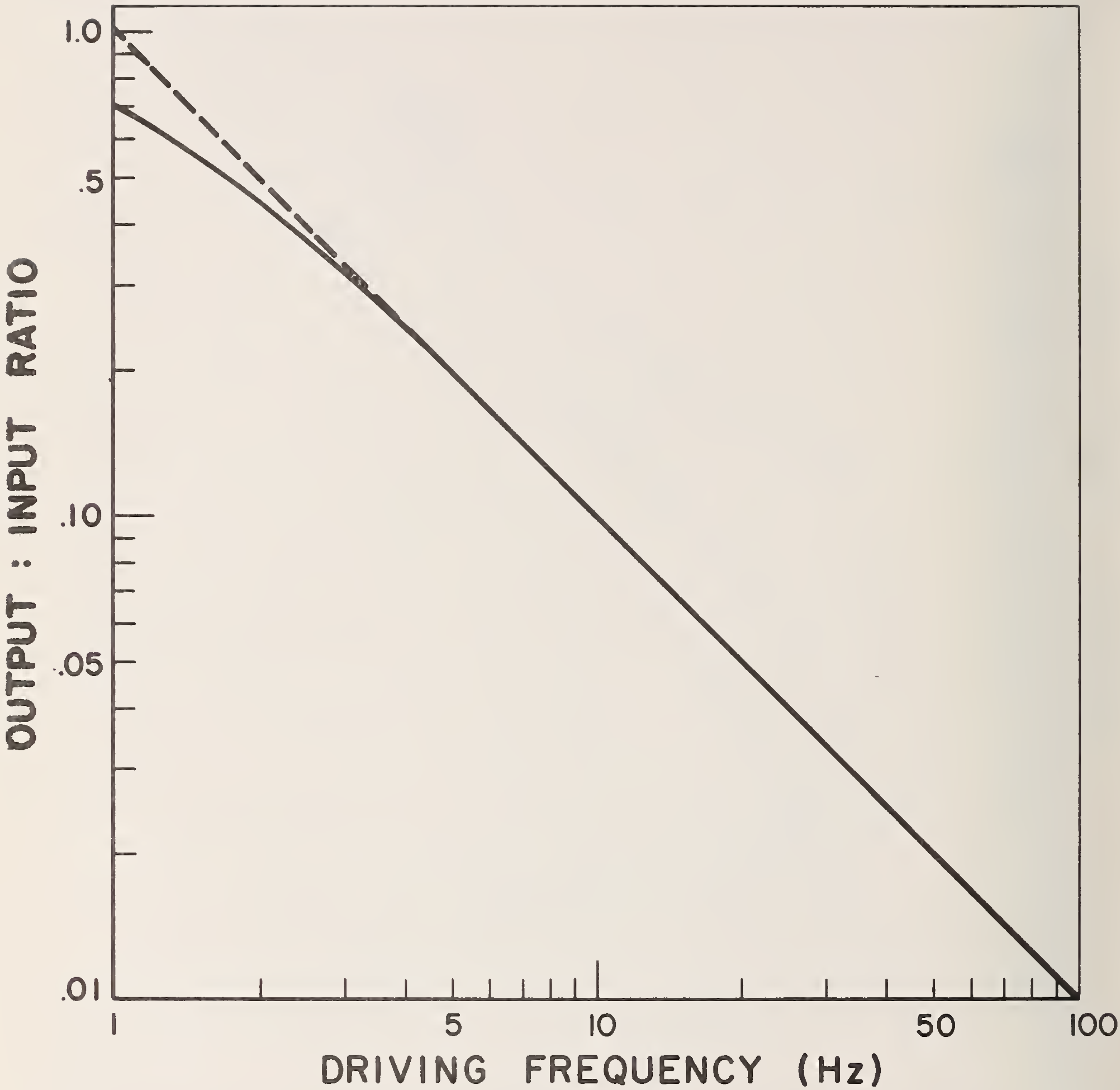


Figure B.9 Output:Input characteristics of a typical 1 Hz integrator

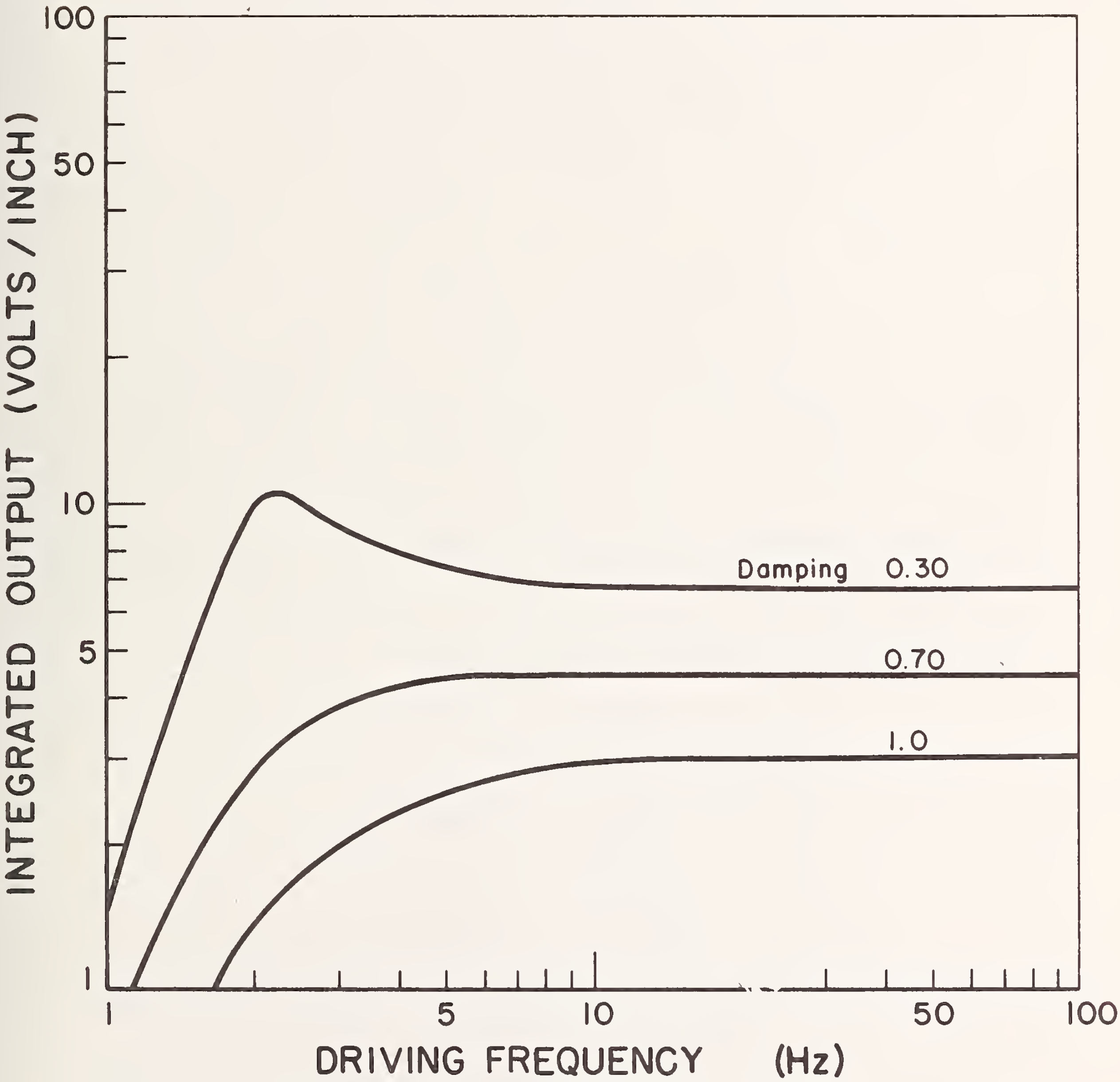


Figure B.10 Displacement response characteristics of the integrated output of a 2 Hz geophone to determine the calibration factor (output per unit of deflection) for the measurement frequency. The integrated output of a geophone is the most common type of motion sensor employed when deflections are measured 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 Figure B.10, 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 (5) using a 16-kip vibrator at three different driving frequencies is shown in Figure B.11. 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.

In 1951 van der Poel (6) 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 dependent upon the driving frequency and at high frequencies S could be expected to increase. 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 (7) presented an equation for calculating 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.

As suggested by several pavement researchers - e.g. Lorenz (8) and van der Poel (9) - a first approximation can be made by representing the pavement structure with the well known case of a mass supported on a

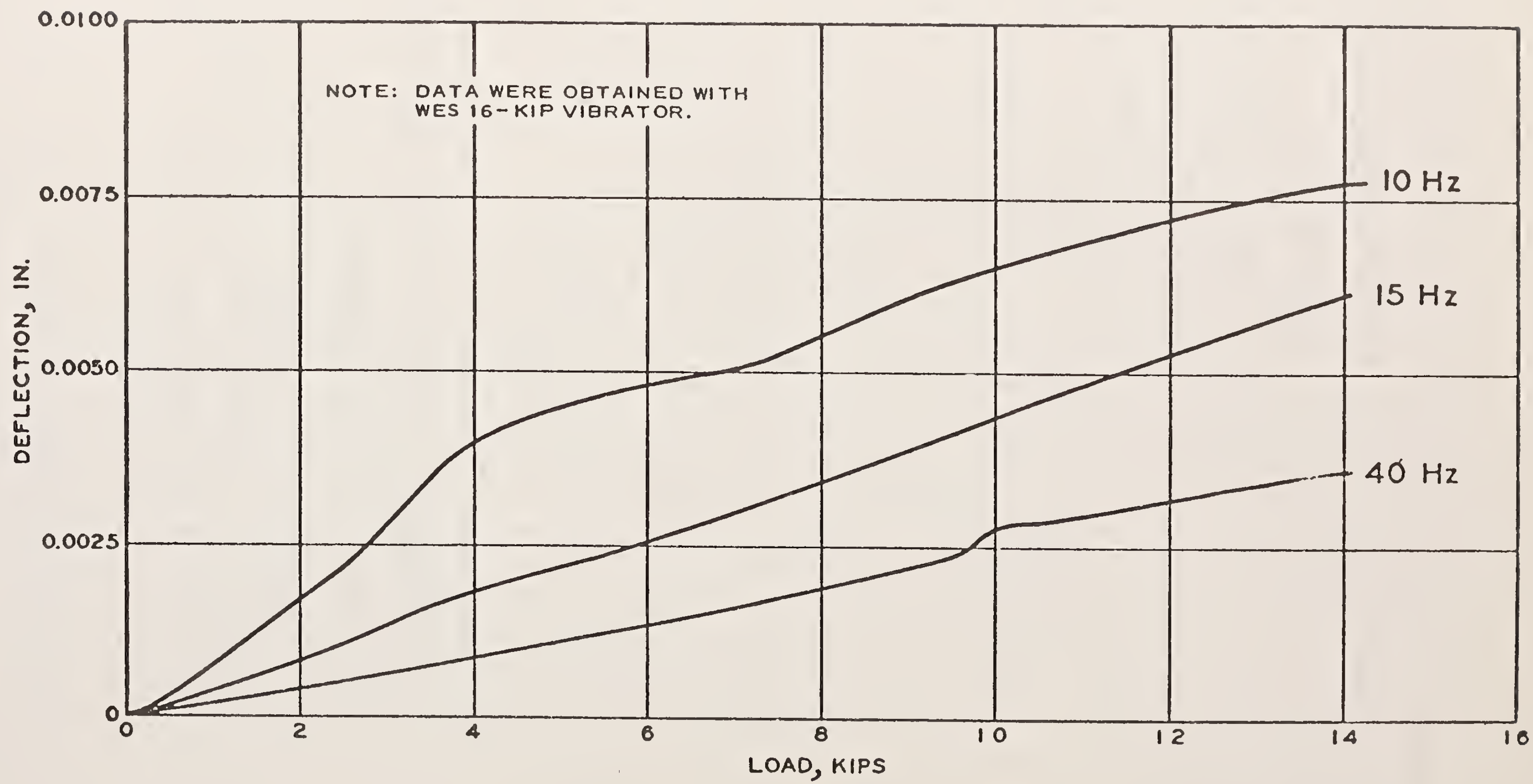


Figure B.11 Typical load-deflection curves for frequencies of 10, 15, and 40 Hz viscously damped spring which is subjected to forced sinusoidal oscillations. A complete analytical treatment of this type of mechanical system which is represented in Figure B.12 can be found in most vibration's texts - e.g. Volterra and Zachmanoglou (10). The force equilibrium equation for this system is

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

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 I is

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

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

$$\phi = \text{arc tan } \frac{2\pi fC}{K - 4\pi^2 Mf^2} \qquad \text{. . . . .(3)}$$

and X_o, the peak amplitude of deflection, is

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

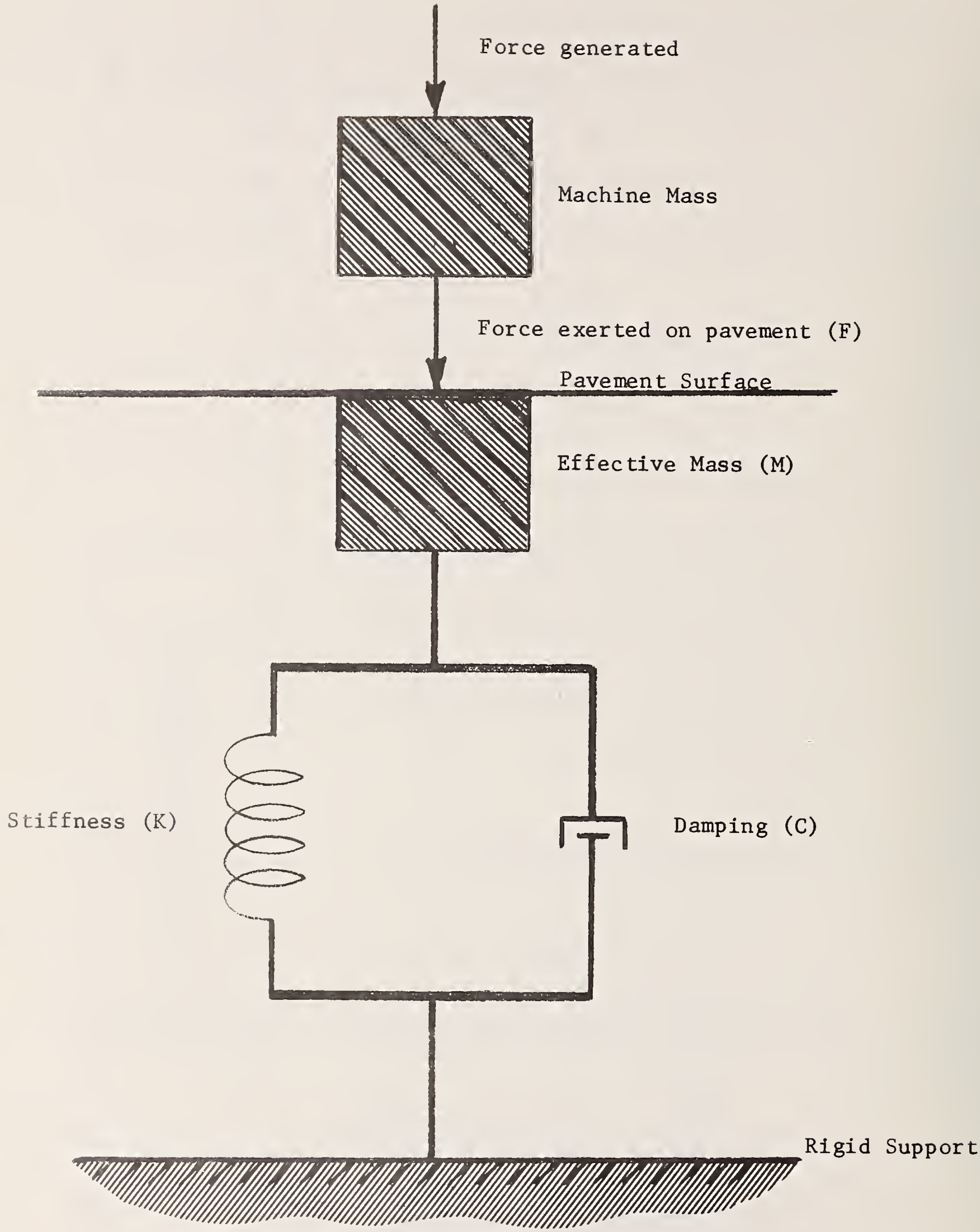


Figure B.12 Mass-spring-dashpot representation of pavement structure subjected to forced dynamic vibrations

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

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

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

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 Figure B.13. 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. In other words the ratio of dynamic to static deflections for equal force approaches unity at low frequencies and may rise or fail to rise at higher frequencies depending on damping.

Based upon their measurements, Heukelom and Foster (11) conclude 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 (12) reported that for all soils

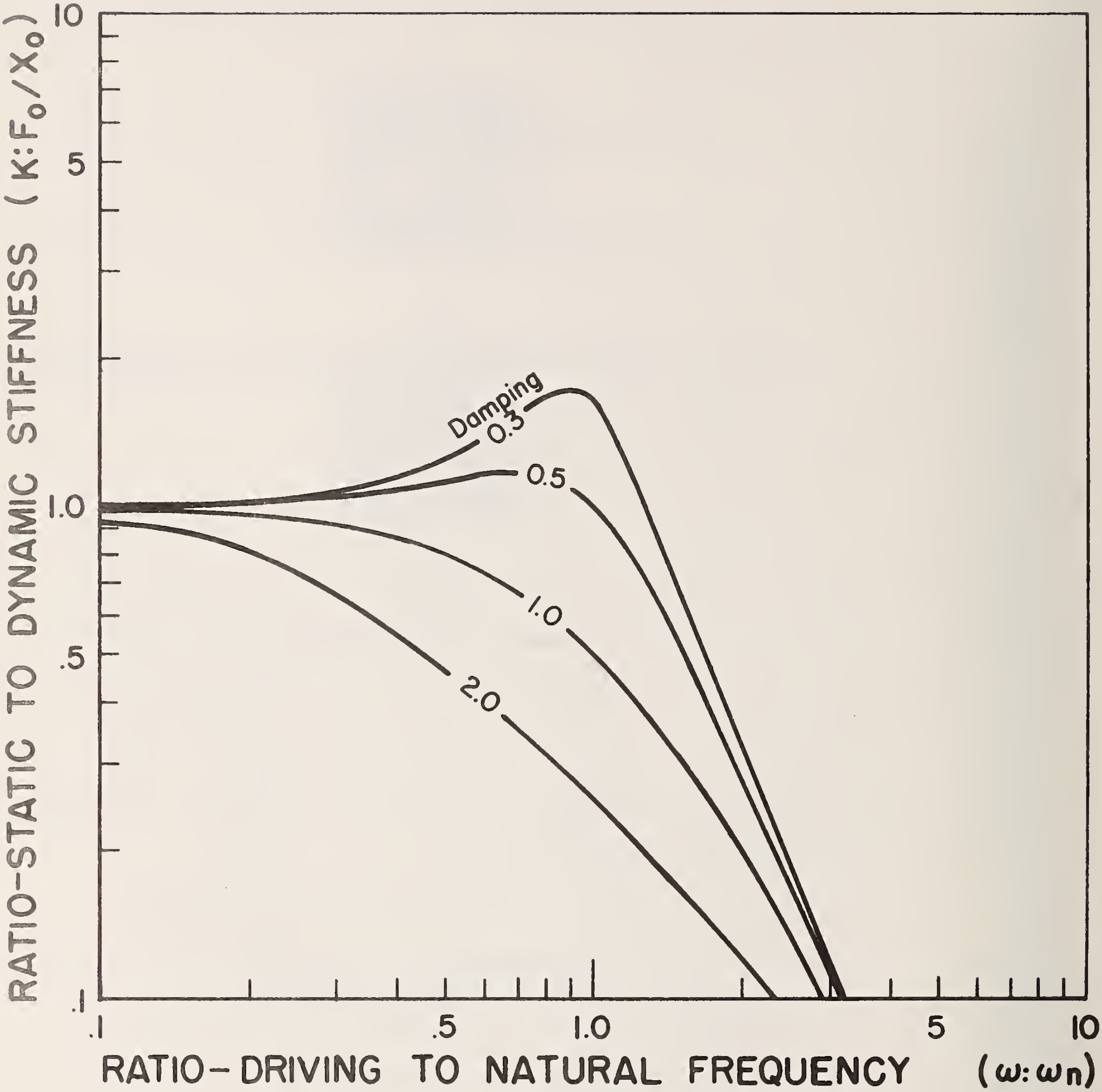


Figure B.13 Response of mass-spring-dashpot system subjected to forced dynamic vibrations and pavement structures that he had observed the approximation was valid until the driving frequency reached 20 to 35 Hz. Szendrei and Freeme (26) have proposed a more sophisticated seven parameter approximation 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 comes 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 the fall of 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 (13). 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 Figure B.14. On either of these pavements it appears that the static response would be obtained at any driving frequency less than about 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 determined using the CXL450 computer program. This finite element computer program was developed in 1970 by Taylor (14) for the analysis of a viscoelastic continuum subjected to oscillatory loading. The thicknesses and material properties assumed in the analysis are given in Table B.1.

Table B.1 Pavement Properties for Viscoelastic Analysis

Layer	Thickness (inches)	Density (pct)	Complex Elastic Modulus (psi)	
			E' - Real	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.

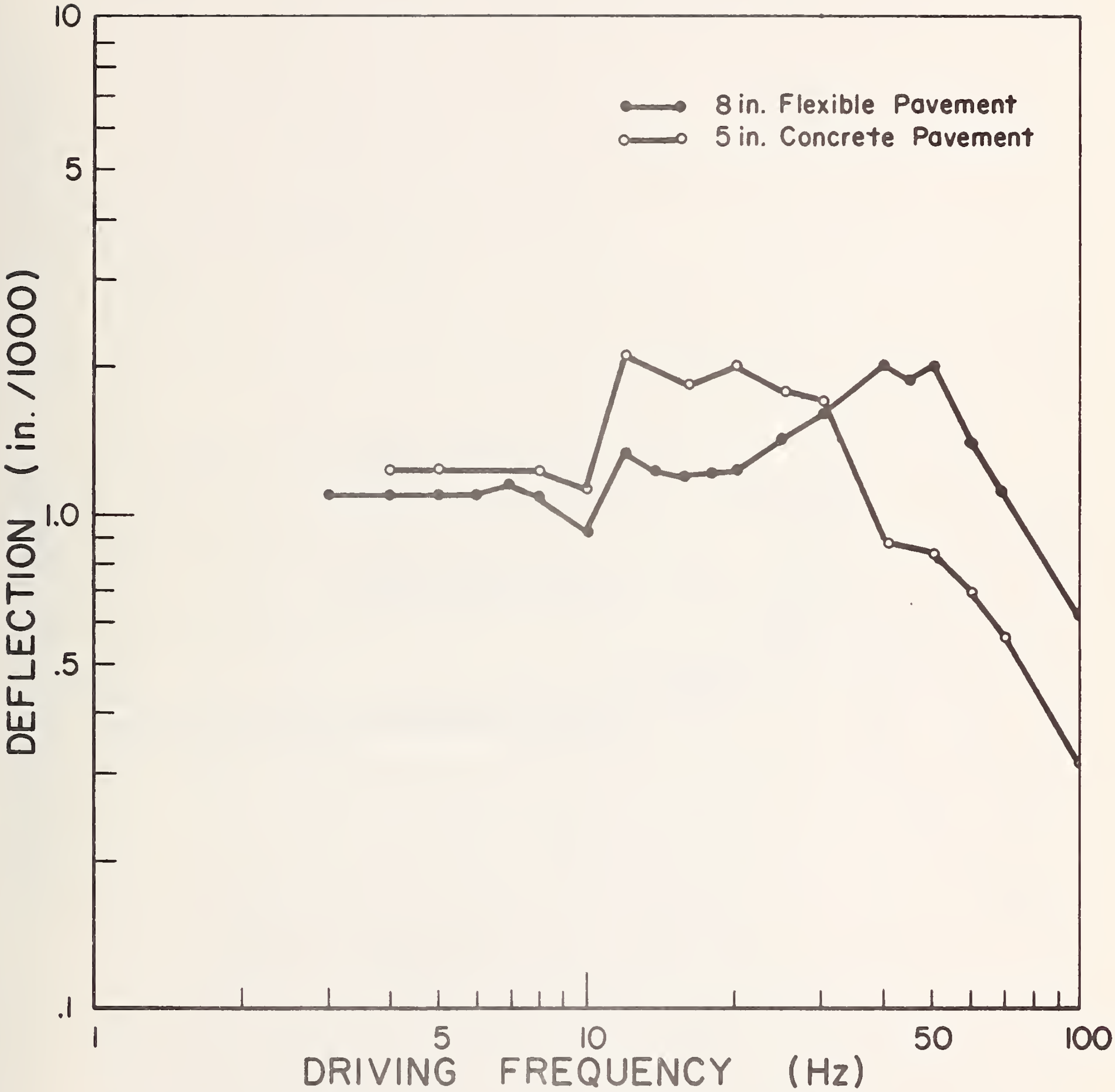


Figure B.14 Measured deflection response of pavements subjected to 1250 lb dynamic force

Figure B.15 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 summary, pavement deflections in response to dynamic loads of 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 the mechanical impedance of a system 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

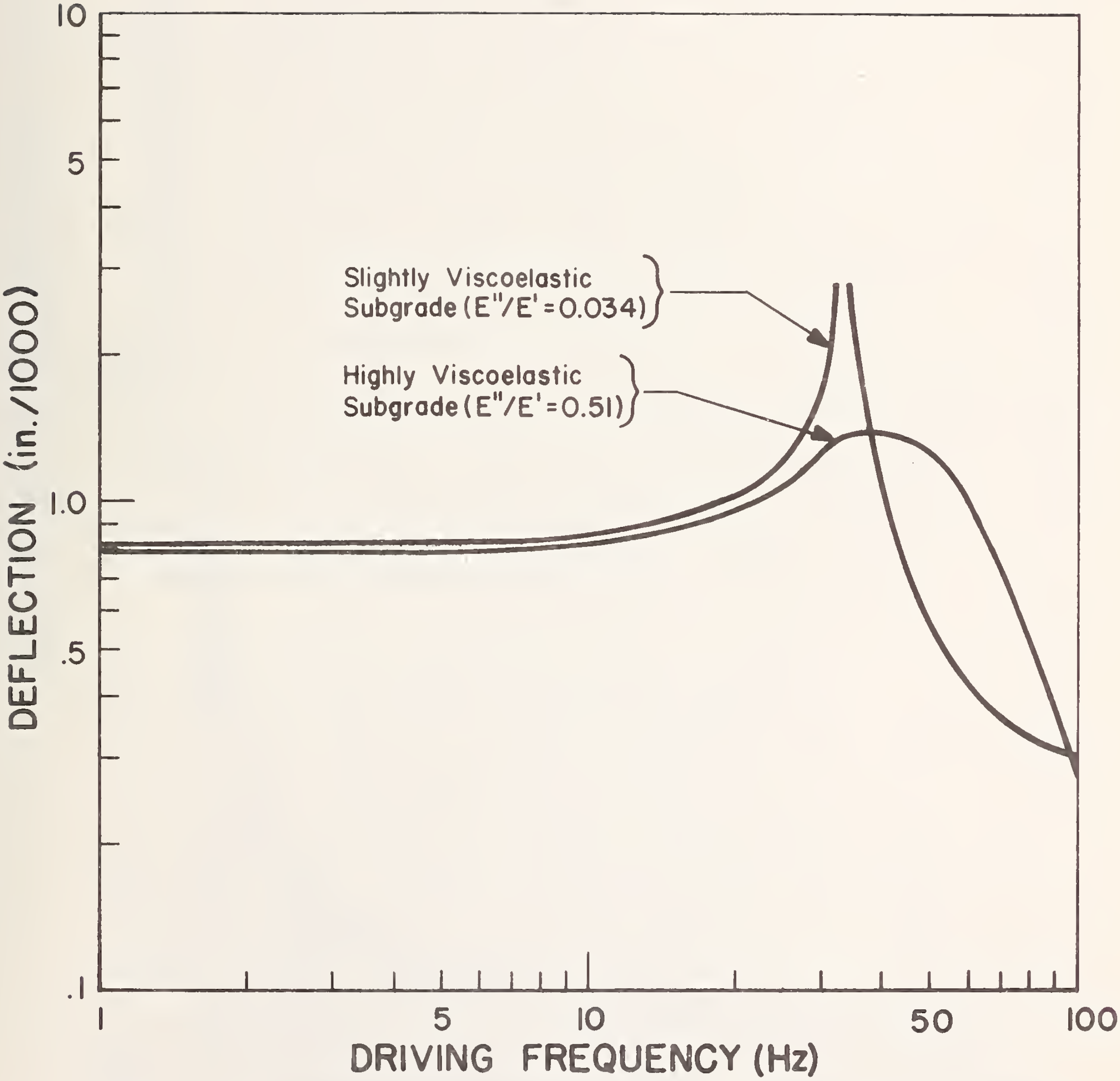


Figure B.15 Theoretical deflection response of three-layered viscoelastic pavement structure the same basic response.

Elastic Response of Pavement Structures

Considerable emphasis has been placed upon determining the elastic properties of the layers in pavement structures. Scrivner et.al. (15) 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 (16) 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.

Figure B.16 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 (17) the deflection basins are the same at large distances from the load. However, in the vicinity of the load, 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 (18). This approach is based upon the Hertz Theory of Plates in the evaluation

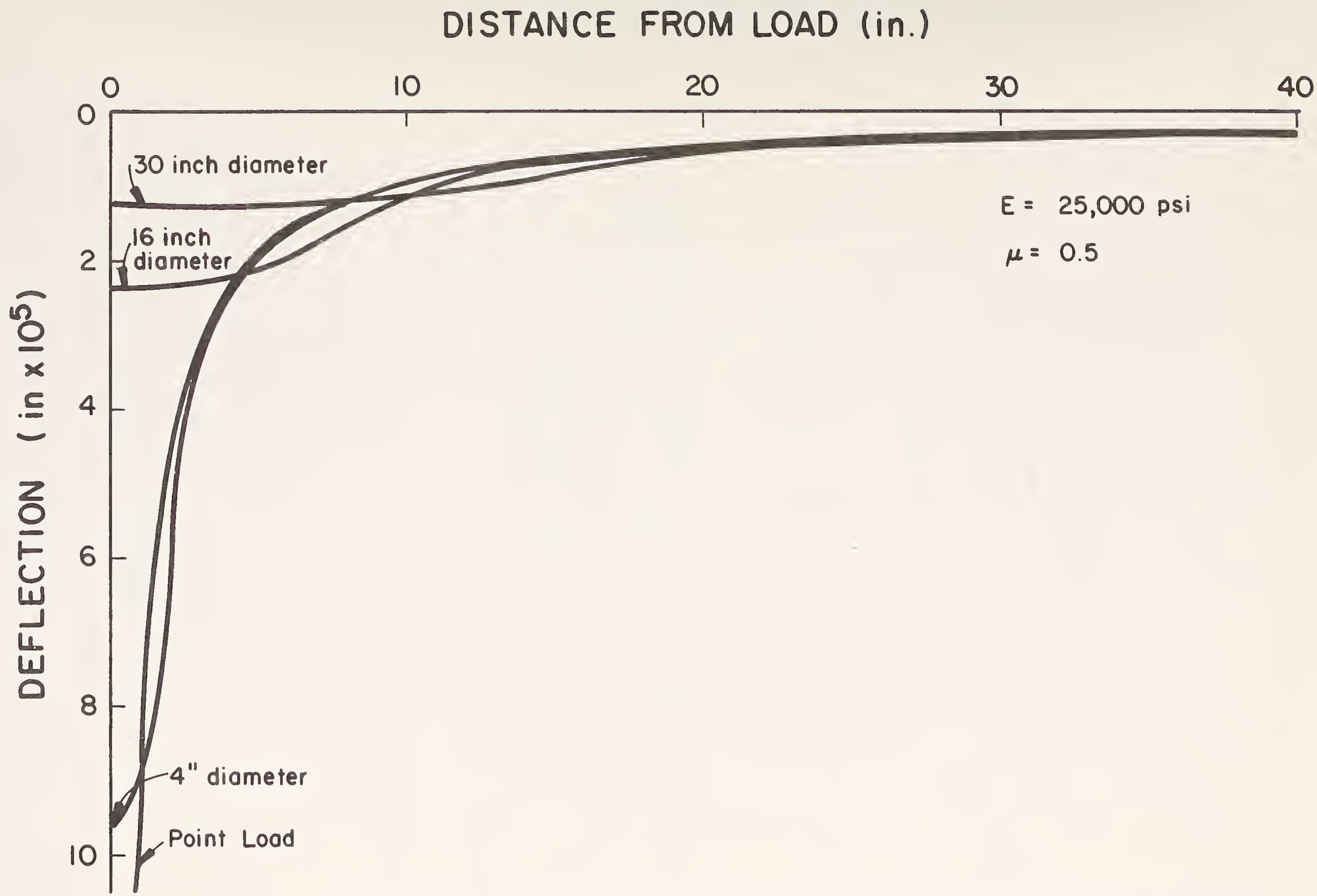


Figure B.16 Elastic deflection basins for 1000 lb load uniformity distributed on circular areas of pavements. Basically it is the application of the analytical solution of a vertically loaded elastic plate floating on a heavy fluid. The solution to this problem was first presented by Hertz in 1884 and 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. Never-the-less 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 elastic layer theory would provide a better fit to measured deflections than 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 St., 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 Figure B.17 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 (19).

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 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 through the meter is recommended to compensate for any possible variations in the sensors and associated circulatory.

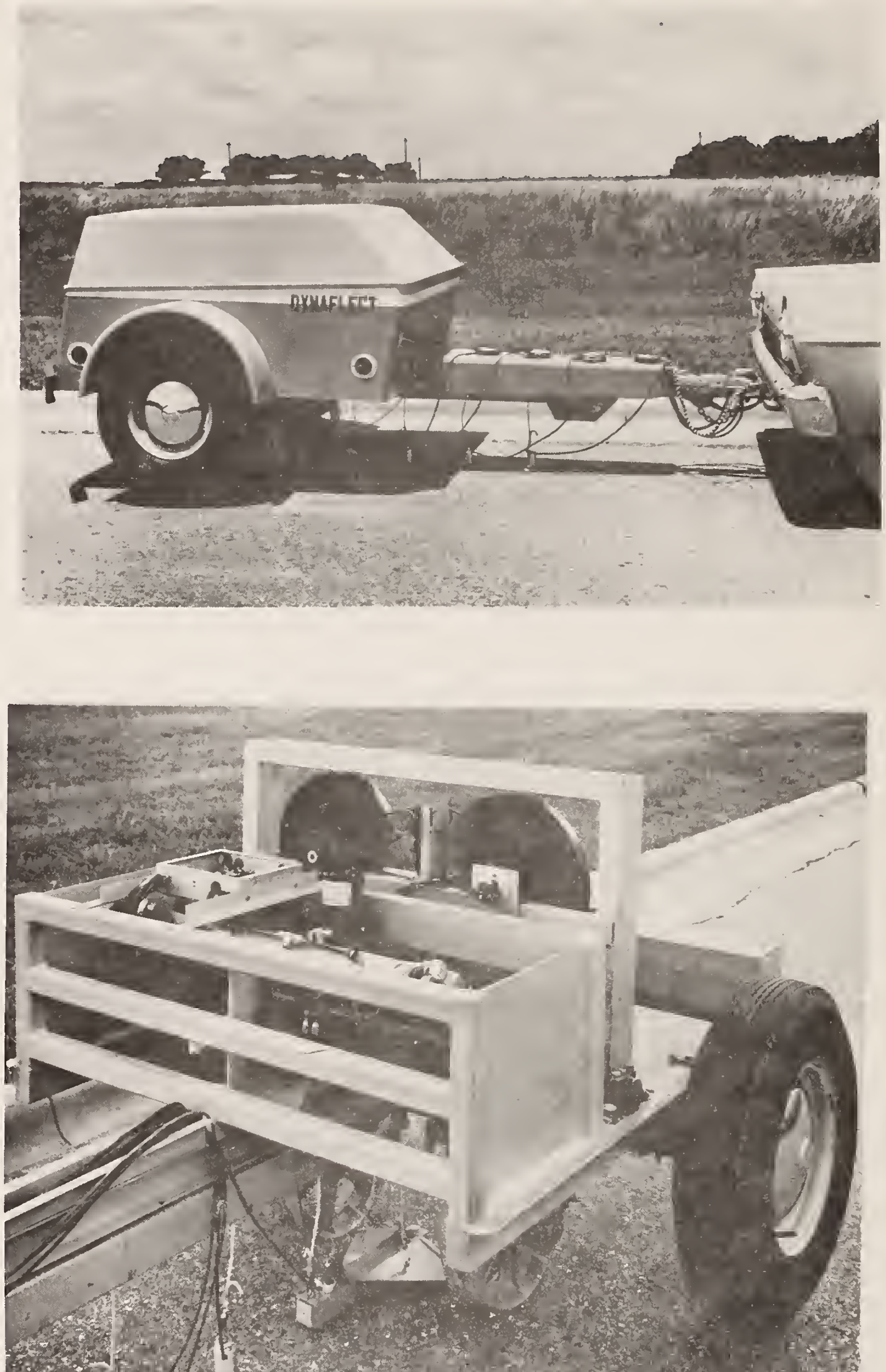


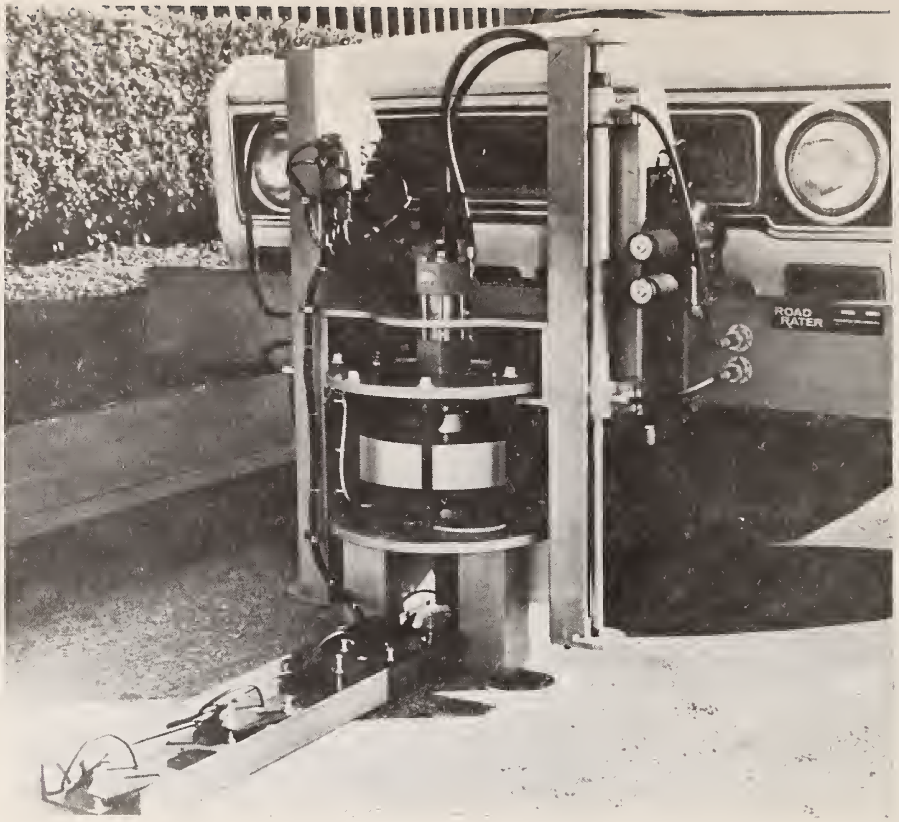
Figure B.17 Dynaflect during measurement operations (upper) and with trailer body removed (lower)

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 Figure B.18). 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 (20).

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 (Figure B.19). 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. in.

Deflections are measured using the integrated output of velocity sensors. Normally one or more sensors are employed to make measurements



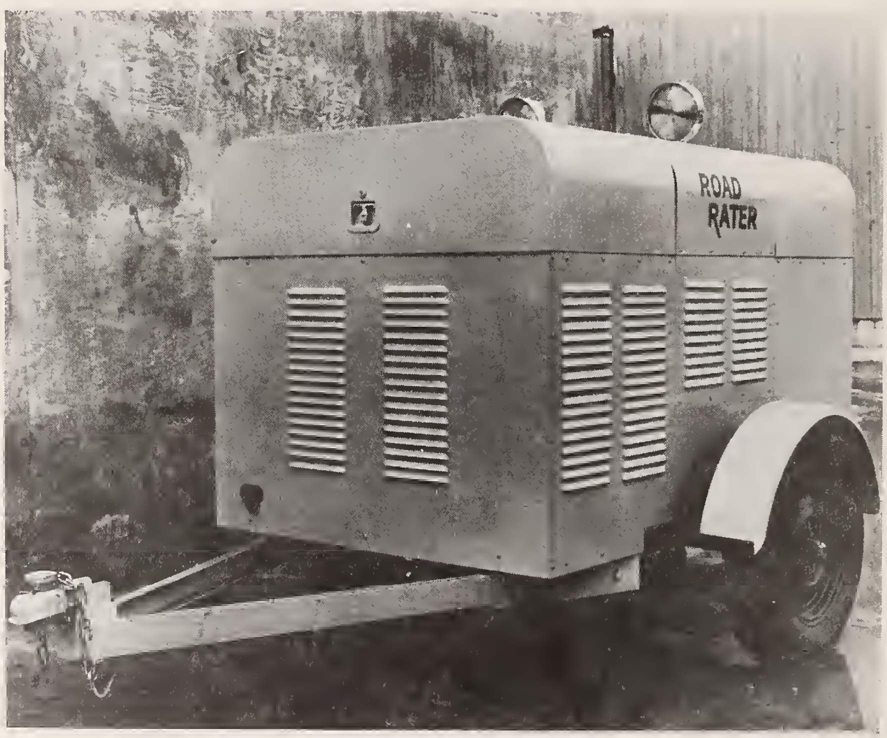


Figure B.18 Road Rater Model 400 (upper) and Model 510 (lower)

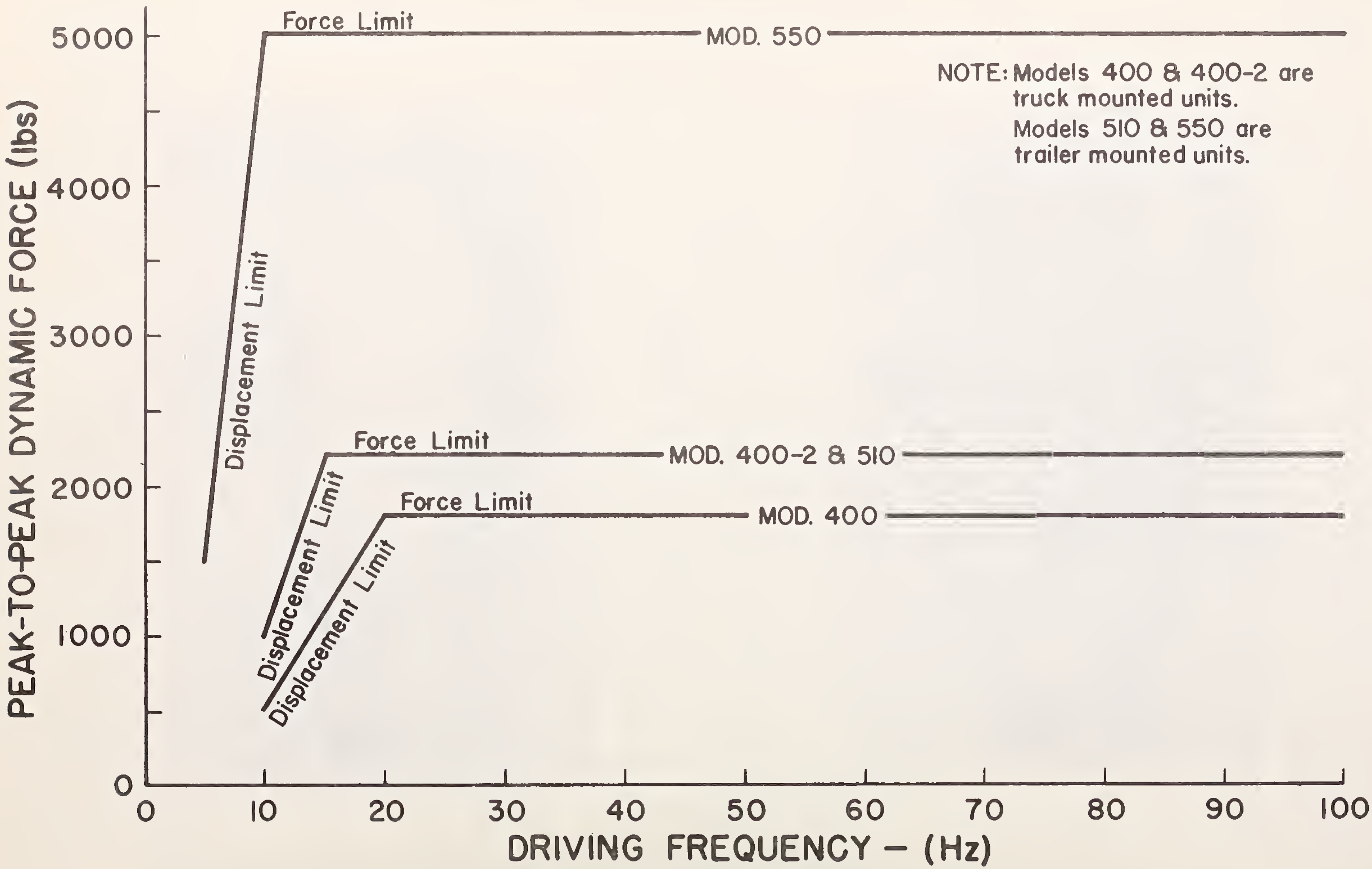


Figure B.19 Dynamic force limits of the Road Rater on the symmetry axis which passes between the two loading plates. The deflection of each sensor is read directly on a meter.

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.

<u>WES 9-kip Vibrator</u>: This instrument shown in Figure B.20 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 (21). To make measurements the truck is stopped at a test location where the force generator is lowered to the pavement through the trailer chasis 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 pre-set so that the peak-to-peak dynamic force is about 16000 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

Figure B.20 WES 9-kip vibrator

Figure B.21 WES 16-kip vibrator

that is shunted to a damping factor of 0.7 and 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 Figure B.21 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. Normally 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 (21).

The force generator consists of an electrohydraulic 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 (22). 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.

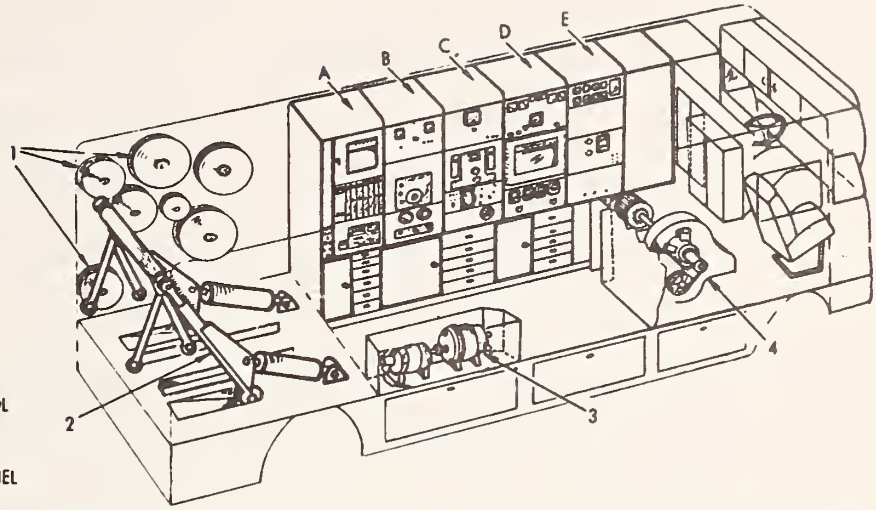
Shell 4-kip Vibrator: This instrument, illustrated in Figure B.22 is mounted in a heavy-duty, panel truck that contains all associated electronics, operation controls and recording equipment (23). A three phase ac generator is also contained in the vehicle which supplies all required power. To make measurements 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 about 8000 lbs. The

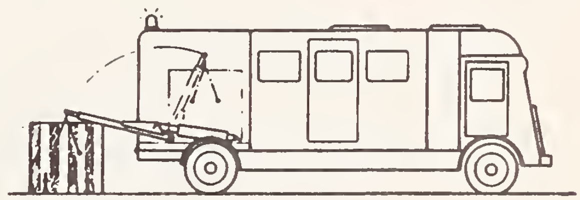
LAYOUT OF INSTRUMENTED TRUCK

1. CABLE REELS
2. HYDRAULIC LIFTING GEAR
3. A.C./D.C. CONVERTER
4. 3-PHASE A.C. GENERATOR SET WITH VOLKSWAGEN ENGINE

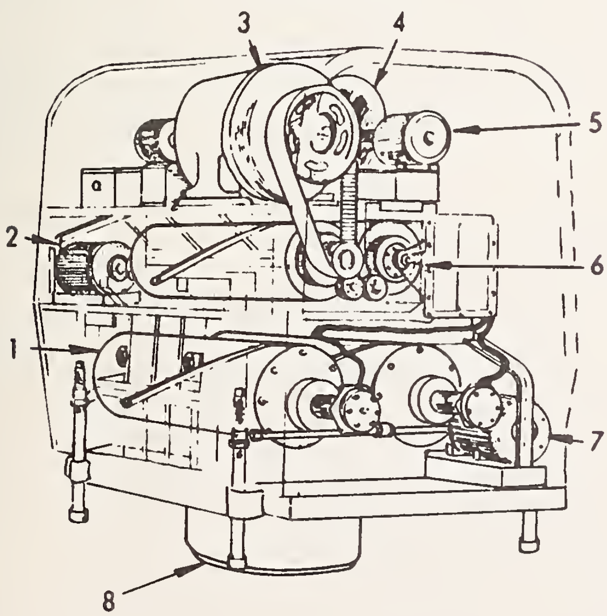
A. TEMPERATURE AND STRAIN RECORDING
B. { WAVE VELOCITY CONTROL
C. {
D. STIFFNESS MEASUREMENT
E. GENERAL INSTRUMENT PANEL



FORCE GENERATOR - LOADING/UNLOADING



FORCE GENERATOR



1. HEAVY VIBRATOR
2. A.C. 3-PHASE, WEIGHT ADJUSTMENT MOTOR FOR LIGHT VIBRATOR
3. LIGHT VIBRATOR MOTOR
4. HEAVY VIBRATOR MOTOR (with belt drive other end)
5. AUXILIARY GENERATOR (for field windings of motor)
6. LIGHT VIBRATOR
7. A.C. 3-PHASE, WEIGHT ADJUSTMENT MOTOR FOR HEAVY VIBRATOR
8. THREE LOAD CELLS MOUNTED IN FOOT OF VIBRATOR

Figure B.22 Shell 4-kip vibrator

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 loads 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 employed by 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 can not be used to detect deterioration in such a layer. Even cracking in such a layer does not appreciably influence deflection 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.

3. Impact Load Response

General

Basically 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 (24) a first approximation can be made by representing the pavement structure with the simple mass-spring-dashpot system illustrated in Figure B.12. A complete analytical treatment of this type of mechanical system can be found in many vibrations texts - e.g. Hansen and Chenea (25). The force equilibrium equation for the system is

$$M \frac{d^2x}{dt^2} + C \frac{dx}{dt} + Kx = f(t) \qquad \qquad \text{.......(1)}$$

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),

$f(t)$ = force as a function of time, and

t = time.

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

$$f(t) = i\,\sigma(t) \qquad \qquad \text{.......(2)}$$

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

$$\int \sigma(t)dt = \begin{cases} 1 \text{ if } t=0 \text{ is included in integration} \\ 0 \text{ if } t=0 \text{ is not included} \end{cases} \qquad \ldots\ldots\ldots(3)$$

the displacement response for the system is given by

$$x(t) = \frac{i}{M\omega_n\sqrt{1-\zeta^2}} \exp(-\zeta\omega_n t) \sin(\omega_n\sqrt{1-\zeta^2}\, t) \qquad \ldots\ldots\ldots(4)$$

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 Figure B.23. 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 by Szendrei and Freeme (26) 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, (27), i.e.,

$$f(t) = \int_{-\infty}^{\infty} F(f) \exp(j2\pi f t)df \qquad \ldots\ldots\ldots(5)$$

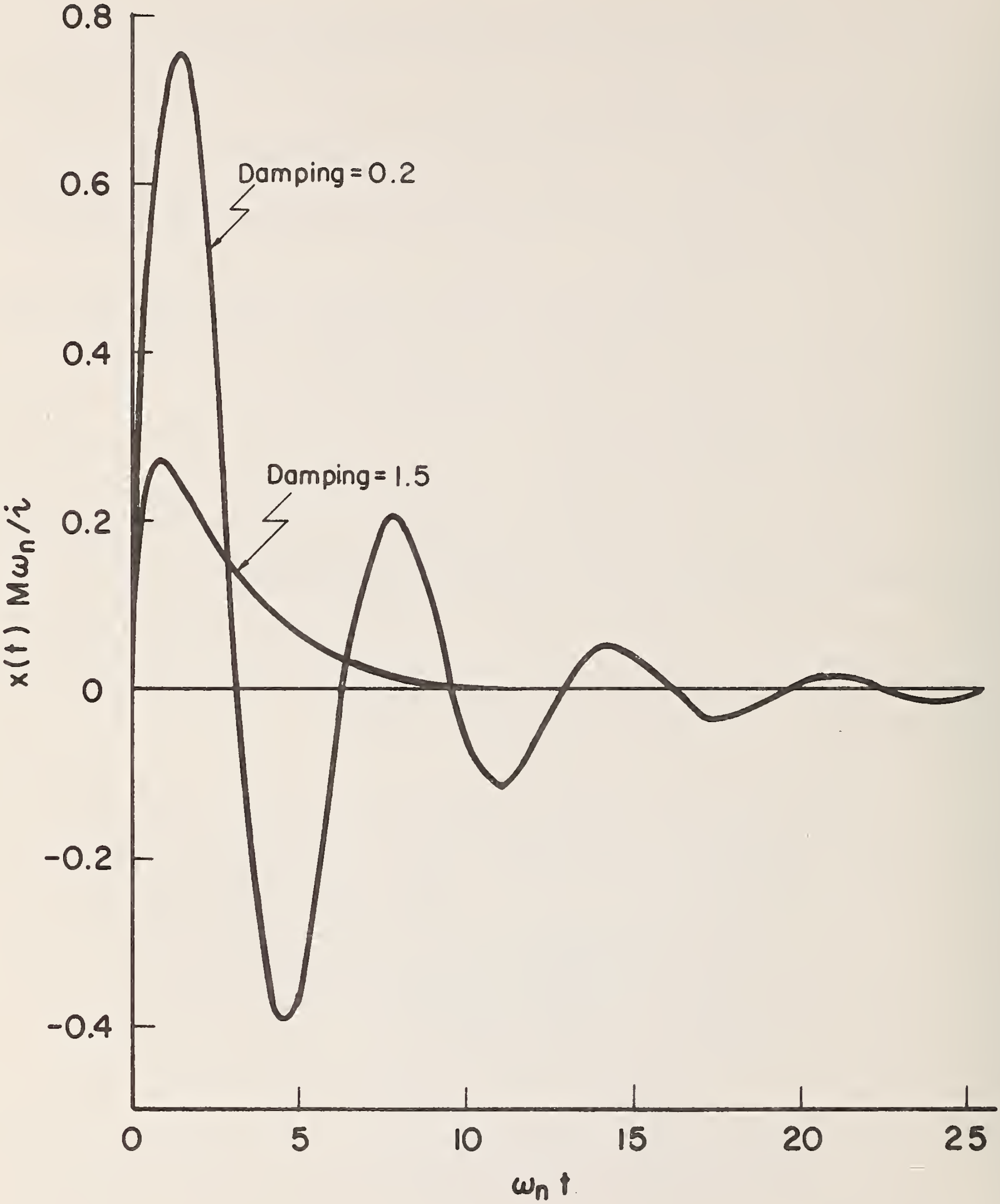


Figure B.23 Response of mass-spring-dashpot system to an instantaneous impulse of force, i

where $F(f)$ is the Fourier transform of $f(t)$ defined by the Fourier integral equation, i.e.,

$$F(f) = \int_{-\infty}^{\infty} f(t)\, \exp(-j2\pi f t)\,dt \qquad \ldots\ldots\ldots(6)$$

$F(f)$ 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 impulsive force, $f(t)$, is related to $X(f)$, through the inverse Fourier transform and 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 output, $X(t)$.

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 2, is (27)

$$F(f) = i \qquad \ldots\ldots\ldots(7)$$

and the Fourier transform of the resultant displacement response defined by equation 4 can be evaluated analytically to be

$$X(f) = \frac{i/K}{1-(f/f_n)^2 + j2\zeta f/f_n} \qquad \ldots\ldots\ldots(8)$$

where

$$f_n = \sqrt{K/M}/2\Pi$$

Equation 8 can be written in the form of magnitude and phase angle as follows

$$X(f) = \frac{i/K}{\sqrt{[1-(f/f_n)^2]^2 + (2\zeta f/f_n)^2}} \quad \text{. . . . .(8a)}$$

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

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

$$\frac{X(f)}{F(f)} = \frac{1/K}{\sqrt{[1-(\omega/\omega_n)^2]^2 + (2\zeta\omega/\omega_n)^2}} \quad \text{. . . . .(9)}$$

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

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. Never-the-less, 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 (26), 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 impulse 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 1960s the Cornell Aeronautical Laboratory, CAL, investigated the feasibility of impulse testing for detecting seasonal variations in the load-carrying capacity of flexible pavements (24). The instrumentation consisted of an impulse generator and several different sensors to measure the pavements response.

The trailer mounted force generator is shown in Figure B.24. 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 wave was made

Figure B.24 CAL impulse generator

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 (28, 29, 30). 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 Figure B.25. 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 Figure B.26, is mounted on a small two wheel trailer that can be towed behind a passenger automobile at normal highway speeds (31). To make measurements the tow vehicle is stopped at a test location where the driver-operator raises the unit from its horizontal transport position into its vertical operationing 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 heights 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

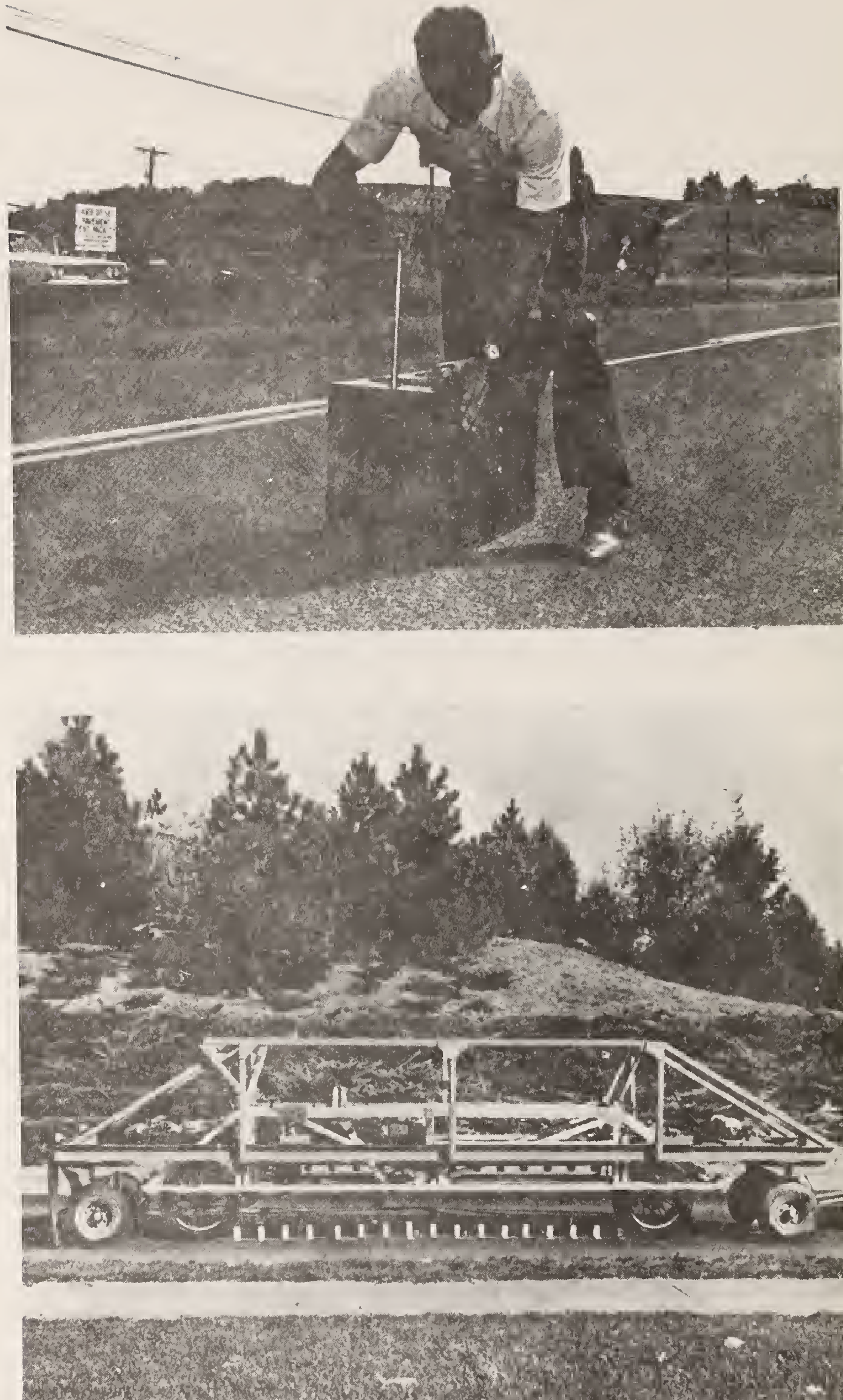


Figure B.25 WSU impulse devices - suitcase model (upper) and vehicle model (lower)

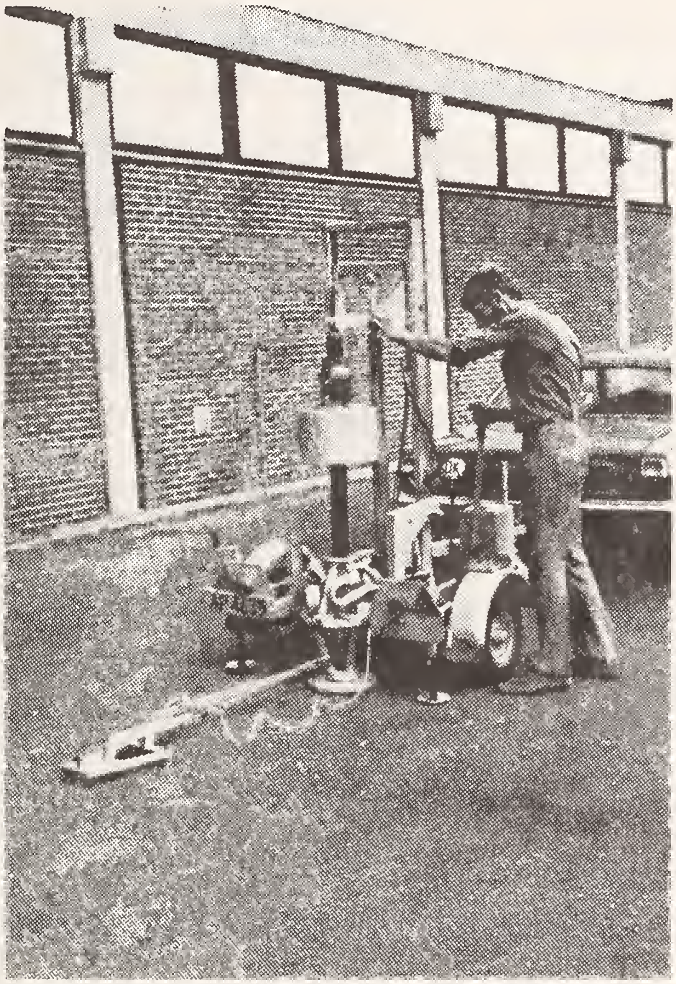


Figure B.26 Phoenix Falling Weight Deflectometer

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 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 volt meter.

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 at most 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. Never-the-less 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.

Basically the impact testing technique is like any type of deflection testing in that it represents characterization of the entire structure. The technique 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.

4. Wave Propagation

General

There are basically two techniques of propagating waves that have been employed for the evaluation of pavement structures. One is the impulsive method. It involves generating an impulse on the pavement surface and measuring the travel time required for the wave fronts that are induced by the impulse to travel through the structure to another location on the surface. The other technique is the steady state method. It involves generating steady state vibrations in the structure and measuring the velocity of the surface wave that propagates away from the vibration source. Although these two measurement techniques are somewhat different, both measure the velocity of elastic waves propagating in the pavement structure in order to estimate the elastic properties of individual layers within it.

Elastic Waves

As a first approximation one can consider a homogeneous isotropic elastic half-space subjected to an external disturbance. In general three different types of elastic waves will be induced that propagate away from the disturbance (32). They are the compressional, shear and surface waves. The compressional and shear waves, also referred to respectively as the dilation and distortion waves or the P and S waves, have spherical wave fronts and will propagate into the medium at different velocities given by.

$$V_c = \alpha_c \sqrt{E/\rho}$$

$$V_s = \alpha_s \sqrt{E/\rho}$$

where

E = Modulus of Elasticity

μ = poissons ratio

$\alpha_c = \sqrt{(1-\mu)/(1+\mu)(1-2\mu)}$

$\alpha_s = \sqrt{1/2(1+\mu)}$

ρ = mass density

The surface wave will not propagate into the medium but will propagate away from the disturbance along the surface of the half space. This wave, commonly called a Rayleigh Wave is similar to the surface wave that is produced when a stone is thrown into water. Lord Rayleigh was the first to investigate surface waves. He surmised that at a large distance away from the disturbance, these waves could be considered as two dimensional plane waves which propagate without much dispersion. Their propagation velocity is given by

$$V_R = \alpha_R \sqrt{E/\rho}$$

where

α_R = material constant dependant upon μ

Values of the elastic wave propagation constants, α, for these three different types of waves are given in Table B.4.

Table B.4

Elastic Wave Propagation Constants for Different Values of Poissons Ratio

μ	α_c	α_s	α_R
0.15	1.028	0.659	0.595
0.25	1.095	0.632	0.582
0.35	1.267	0.609	0.569
0.45	1.948	0.587	0.557
0.50	∞	0.577	0.552

Rayleigh waves like compressional and shear waves travel at a constant velocity which is independent of their frequency. However, Rayleigh waves attenuate very rapidly with depth. The attenuation is directly proportional to the wave length of the surface wave. In fact the wave motion becomes insignificant at a depth of a few wave lengths (33). If an external disturbance is generated which is predominantly composed of a high frequency component, it will generate a surface wave with a short wave length which will not penetrate very deeply. Whereas, an external disturbance composed predominantly of a low frequency component, will generate a surface wave with a long wave length which will penetrate much deeper.

Impulse Waves: When a pavement structure is subjected to a brief impulse, both compressional and shear wave fronts are propagated into the structure. As either type of wave front reaches an interface between two pavement layers it is reflected and refracted into both compressional and shear wave fronts. In other words, a single wave front produces four new wave fronts in the structure when it reaches an interface. Thus, in multi-layered pavement structures the propagation of elastic waves becomes a very complex phenomenon. Never-the-less, it is sometimes possible to identify the nature of particular wave fronts at detection points on the surface and thereby deduce elastic wave velocities.

When a pavement structure is subjected to a brief impulse, the response at any detection point on the surface is the superimposition of the wave fronts that travel through the structure in all conceivable wave paths to the detector. With current transducers it is difficult to accurately define the arrival of any wave front except the one that arrives first. The predominant impulse testing that has been reported for pavement structural evaluation has been based upon determinations of the earliest arrival which is surmised to be the compressional wave.

When a brief impulse is produced, the first arrival near the disturbance can safely 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 also possible to deduce the compressional wave velocity in the second layer. The technique for such determinations has been described previously (34). Basically 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 this compressional wave front occurs before the arrival of compressional wave front that is traveling directly through the top layer. Portable seismographs like the Geo Space Model GT-2, manufactured by Geo Space Corporation of Houston, Texas and the Terra-Scout Model R15, manufactured by Soil Test, Inc., of Evanston, Illinois have employed this technique to locate buried rock prior to pipeline trenching operations.

If the arrival of several of the wave fronts could be accurately timed at various points on the surface, it should be possible to determine their travel paths and thereby deduce their propagation velocity in some of the individual layers. Not much success has been reported in the literature through use of this approach. The principle difficulty is that the induced waves do not have sharp arrivals and the transducers can not readily distinguish between a new front arrival and the tail of a proceeding front.

Steady State Waves: When a pavement structure is subjected to a steady state disturbance, wave front arrivals no longer occur. In fact the whole structure is set into continuous vibration at the frequency of the disturbance. The vibrations at any point away from the disturbance, however, occur at a slightly delayed time or at a lagging phase angle with respect to the disturbance. Because surface waves propagate with much less dispersion than waves that propagate into the media, their velocity of propagation is readily determinable on the surface of the pavement. If high frequency disturbances are generated, surface waves can be produced that have a wave length that is short compared to the surface layer. Such surface waves do not penetrate deeply and propagate at the velocity of a Rayleigh wave in the material of the surface layer. On the other hand, if low frequency disturbances are generated, surface waves can be produced that have a wave length that is several times the thickness of the pavement structure overlying the subgrade. Such surface waves will penetrate the entire structure and will propagate at a velocity of a Rayleigh wave in the material of the subgrade. Between these two extremes the surface wave will travel at various velocities dependent upon the thicknesses and elastic properties of the various layers in the structure.

Generally for interpretation a plot is made of the surface wave velocity versus its wave length. This plot is called a dispersion curve. The Rayleigh wave velocity of the surface material is assumed to be the velocity when the curve is projected to zero wave length. The Rayleigh wave velocity of the subgrade material is assumed to the asymptote velocity of the curve projected to very long wave lengths. To interpret the remainder of the curve various assumptions have been made to calculate theoretical dispersion curves for multi-layered structures. Several theoretically deduced dispersion curves representing some typical pavement structures were presented by Jones in 1962 (35). One of them, a two-layer system of a plate overlying a material having a relatively low shear modulus is illustrated in Figure B.28. On this figure the Rayleigh wave velocity of both the surface layer and the bottom layer are shown.

Additional work has been done to better understand and interpret dispersion curves for pavement structures developed from steady state wave propagation testing. In July 1974, an excellent review of this work was presented by Watkins, Lysmer and Monismith (36). Much of their work has followed the fundamental approach. The solution to the general differential equations of motion for layered elastic structures are obtained for various assumed boundary conditions. The interpretation of data, therefore, depends largely upon the ability to develop theoretically deduced curves for layered structures which compare to measured data. This is an extremely complex process. Even with the most sophisticated equipment and data acquisition program, many uncertainties remain.

Testing Equipment

There are many variations in commercially available equipment which

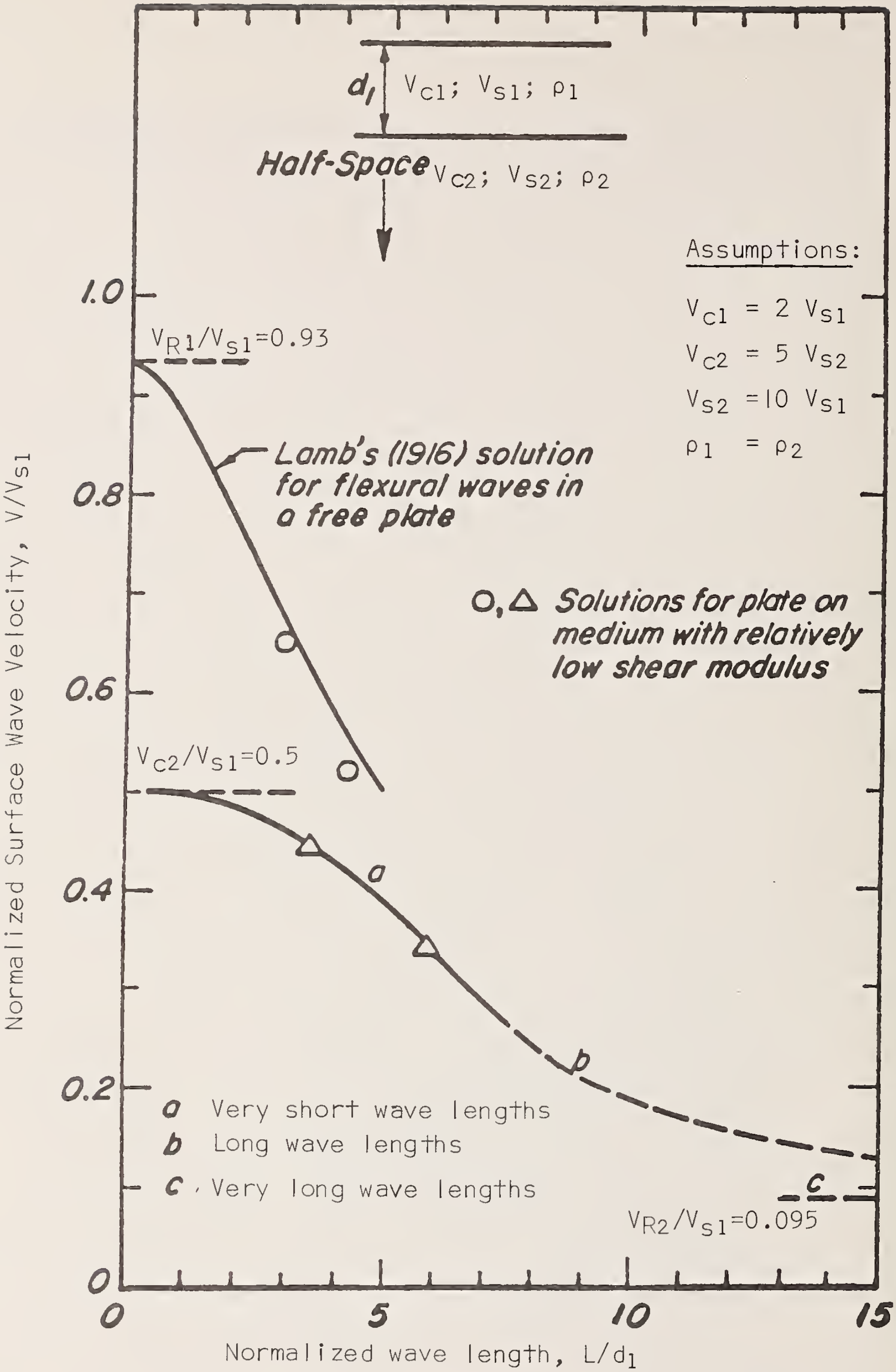


Figure B.28 Theoretical dispersion curve developed by Jones (35) for a plate overlying a material of low shear modulus can be used for investigating impulse wave propagation in pavement structures. Two different techniques for impulse generation and measurement are described in this section.

Although variations of testing equipment are used to investigate steady state wave propagation in pavement structures, generally the equipment differences are not great. The method of developing dispersion curves are generally based upon the testing procedures developed by researchers at the Royal Dutch Shell Laboratory in Amsterdam and the Road Research Laboratory in England. The measurement technique developed by them is described in this section.

Hammer Impulse: This measurement technique is largely based upon a somewhat similar technique of subsurface exploration developed by seismologists. It consists of delivering an impulse to the surface of a pavement by striking it with a hammer. The resultant ground motion at one or more points on the surface is observed with geophones.

Normally horizontal motion geophones are used instead of vertical motion geophones which are conventionally used by seismologists to detect reflections from deeply buried stiff layers. Horizontal motion geophones respond much better to the motion of the fast compressional wave traveling along the surface. Its motion is predominantly horizontal.

The output of a geophone is observed on a timing oscilloscope which has been triggered by the hammer blow. Through such observations it is possible to measure the time required for the compressional wave to travel through the surface material to the detection point. The distance from the impulse point to the detection point divided by the measured time is taken to be the compressional wave velocity. Figure B.29 illustrates the typical equipment used for hammer impulse testing. The upper photograph shows the hammer with its triggering cable and the lower photograph shows a received signal on a timing oscilloscope.

Transducer Impulse: This measurement technique is similar to the hammer impulse technique. The chief difference is that an impulse is generated electronically with an acoustic transducer. Such transducers are inherently resonant devices that produce and receive impulses having the form of lightly damped wave trains that oscillate numerous times while building up to their maximum amplitude. For accurate timing it is desirable to have the frequency of this wave train as high as possible. However, the granular nature of pavement materials cause scattering and attenuation of waves whose length is comparable to, or shorter than, the size of the larger aggregate particles. This phenomenon generally sets the maximum desirable transducer frequency to be about 50 KHz.

Velocities are measured employing about the same procedure described for a hammer impulse. A receiving transducer is placed on the surface a short distance away from the impulse (or transmitting) transducer. The output of the receiving transducer is monitored on a timing oscilloscope which has been triggered at the same instant of initiation of the impulse. The distance between the transmitting and receiving transducers divided by the time is taken to be the compressional wave velocity. Figure B.30 shows a device developed by Swift and Moore (37) for the measurement of compressional wave velocity in concrete slabs.

Figure B.29 Hammer impulse wave propagation

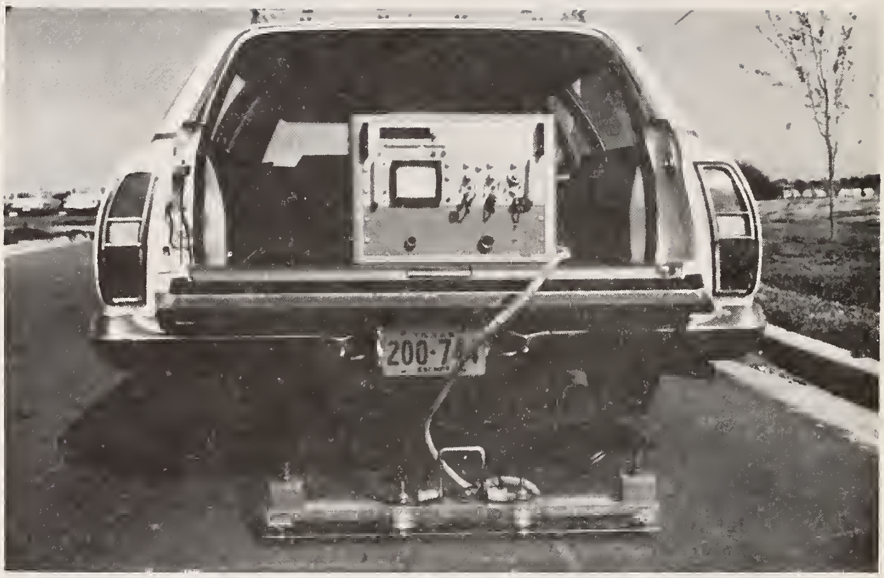


Figure B.30 Transducer impulse compressional velocity measuring instrument designed for use on concrete slabs

Steady State Vibration: The measurement of steady state wave propagation in pavement structures has been pioneered by researchers of the Royal Dutch Shell Laboratory as well as of the Road Research Laboratory in England. This work is well documented in the literature. See for example Heukelom and Klomp (38) and Jones, Thrower and Gatfield (39). Basically the procedure employed by them is to measure the wave length of the surface waves that propagate away from a steady state vibration generator which is placed on the surface of the pavement. When the wave length is measured for any value of driving frequency the velocity of propagation is the product of the driving frequency times the wave length.

In practice the measurements are obtained for a wide range of driving frequencies in the range of about 35-4000 Hz. At any driving frequency a geophone pick-up is moved progressively away from the vibrator and successive positions are found at which the vibrations are in phase with the vibrator. The distance between two successive positions is equal to the wave length. However, generally a plot is made of the distance of several such positions so that a more reliable average wave length measurement can be obtained.

Almost all steady state wave propagation investigations have been done with small electromagnetic vibrators. However, some of the large force generators described previously in Section 3, Steady State Dynamic Deflections, Testing Equipment, have been used for low frequency investigations between 10 and 100 Hz. The small electromagnetic vibrators normally employed are driven with a power amplifier and an oscillator. They can normally be operated at any driving frequency from about

35 to 4000 Hz by simply setting the oscillator. The oscillator signal is also used to generate a sharp pulse or phase mark for referencing the phase of the geophone pick-up to the phase of the vibrator.

Figure B.31 shows typical equipment used for steady state wave propagation investigations. The upper photograph shows the electromagnetic vibrator and the geophone pick-up in operation and the lower photograph shows the instrumentation used for driving the vibrator and observing the output of the pick-up.

Measurements are normally made over the entire frequency range and complete dispersion curves are developed in the field. At the higher frequencies short wave length surface waves are produced which are primarily influenced by the surface layer. At the lower frequencies long wave length surface waves are produced which are primarily influenced by the subgrade. By developing a complete dispersion curve in the field, it is possible to explore in detail any discontinuities in the plot which is normally considered to represent a change in the mode of wave propagation.

Application

Neither the impulse nor the steady state wave propagation techniques have been developed sufficiently for general application by highway engineers in the non-destructive evaluation of entire pavement structures. However, within current technology equipment can be designed which can measure the compressional wave velocity in the surface layer. Such equipment can be made to be relatively fast and inexpensive like the instrument illustrated in Figure B.30.

In general it will be necessary to make further research advances

Figure B.31 Steady state wave propagation before impulse wave propagation can be implemented for more sophisticated applications than the one described above. For example, in the metals testing industry it is currently possible to construct transducers that produce relatively narrow beam elastic waves which can be directed into or along the surface for specific testing applications. These transducers produce waves which have frequency components in their wave trains that are much too high to be suitable for pavement materials. As mentioned previously it is necessary to employ waves which have a wave length larger than the size of the coarse aggregate particles to prevent rapid scattering and attenuation. Research in the development of improved transducers may yield similar practical benefits. In addition the swept frequency "Vibroseis" technique developed by Continental Oil Company appears to offer promise in its application to pavement evaluation. The advantage of this technique is that it permits very accurate timing of extremely weak wave front arrivals.

Further research advances will also be required before steady state wave propagation techniques can be implemented for practical use. At present such developments do not appear very promising. In fact, most of the active research work along this line has been discontinued in the United States. There are well known research agencies in other countries namely in England, Australia, and South Africa who are actively investigating this approach.

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