NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **59**

STANDARD MEASUREMENTS FOR SATELLITE ROAD TEST PROGRAM

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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STANDARD MEASUREMENTS FOR SATELLITE ROAD TEST PROGRAM

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
PAVEMENT DESIGN
PAVEMENT PERFORMANCE
GENERAL MATERIALS
FOUNDATIONS (SOILS)

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1968

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

NCHRP Project 1-6 FY '64

NAS-NRC Publication 1713

Library of Congress Catalog Card Number: 70-600028

FOREWORD

By Staff

Highway Research Board

NCHRP Project 1-1 established the measurement program that is considered minimal for a nationwide coordinated satellite program, but it did not include specifications for equipping, staffing, training, and operating teams for making "common denominator" measurements on the various satellite projects. Such specifications have been developed in the project reported herein (NCHRP Project 1-6). Although a nationwide satellite road test program has not materialized on the scale envisioned at the completion of the AASHO Road Test, conducted at Ottawa, Ill., a number of states have proceeded independently to adapt Road Test findings to a variety of materials and climatic conditions, and they, in particular, will find the results of this project to be of direct usefulness for the reasons that (1) it establishes the feasibility and procedures for operation of a measurements team for a satellite road test program; (2) recommendations are made for standard sampling and testing of materials and soils for pavement test sections, regardless of whether or not they are included in a satellite program; (3) procedures are described for modifying the determination of present serviceability index of pavements to include a term for surface texture; and (4) an evaluation is included of test data obtained from experimental pavement sections built during this project.

The structural design of highway pavements involves empirical techniques based to a large degree on long experience of highway agencies augmented by test programs, the most ambitious of which was the AASHO Road Test conducted near Ottawa, Ill., and completed in the fall of 1960. Due to the fact that a field test program involving the many variables known to affect pavement performance would become so complex and expensive as to become unfeasible, the sponsors of the Road Test chose to include only a limited number of variables in the project. As a result, it is generally recognized that the findings of the study relate only to the conditions at the Road Test site. Applications of these findings in other areas of the country should be based on experimental or other evidence of the effects of differences in subgrade soil, materials, construction practices, traffic, environment, and maintenance procedures.

Following completion of testing, it became necessary to analyze and evaluate the large volume of data and to determine maximum utilization of the findings in terms of application to pavement design procedures. The most significant aspect of this problem was that of selecting a method for extending or translating the Road Test findings from the particular conditions of the test site to the soil, environmental, and materials conditions of the various areas of the country. One approach to providing a means for early application of project findings was the incorporation of a soil support value and a regional factor into pavement design procedures developed from the Road Test data and distributed by the AASHO Committee on Design as the "AASHO Interim Guide for the Design of Flexible Pavement Structures" and the "AASHO Interim Guide for the Design of Rigid Pavement Structures." Another approach being envisioned at the conclusion of the Ottawa study was the establishment of a nationwide satellite road test program consisting of individual studies on existing or newly constructed pavement test sections throughout the country.

On the basis of the satellite road test concept, NCHRP Project 1-1(1), "Development of Procedures for Comparing the AASHO Road Test Findings with Performance of (1) Existing Pavements and (2) Newly Constructed Experimental Pavements," was initiated, with the Highway Research Board as the research agency. The project report was published as *NCHRP Report 2,* "An Introduction to Guidelines for Satellite Studies of Pavement Performance," and *NCHRP Report 2A,* "Guidelines for Satellite Studies of Pavement Performance." These reports emphasize that meaningful results from such studies will depend on the ability to correlate information between studies and with the original AASHO Road Test, and they recommend the establishment of measurement teams equipped and trained to make "common denominator" measurements on satellite projects to supplement data collected by the individual sponsors.

The essential objectives of the Texas Transportation Institute in carrying out NCHRP Project 1-6, "Standard Measurements for Satellite Program—Measurement Team," were to determine the feasibility and estimates of costs for operation of such teams. This publication constitutes the final report of the project on the accomplishment of the objectives. A firm basis for equipping, staffing, training, and operating teams for making common denominator measurements has been established. Consequently, those agencies conducting individual field studies and other research aimed at extending the Road Test findings to their particular conditions will find the results of the study to be directly useful. Other uses will be found because of the general information it contains that is applicable to materials testing and pavement design. Of particular interest is the information contained in Appendix F, pertaining to the use of the Lane-Wells Dynaflect equipment as a nondestructive means for estimating the relative stiffness of individual layers of a flexible pavement section.

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ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 1-6 by the Pavement Design Department, Texas Transportation Institute, Texas A&M University, with F. H. Scrivner, Research Engineer, as principal investigator. He was assisted in the research and the preparation of the report by W. M. Moore, Assistant Research Engineer; and by M. B. Phillips and Rudell Poehl, who authored Appendices E and F.

Grateful acknowledgment is extended to the following individuals and agencies who contributed to the research work performed in the project or assisted in the preparation of the project report:

The Texas Highway Department and the Bureau of Public Roads, who shared, with the Highway Research Board, the cost of constructing the Pavement Test Facility described in Chapter Ten.

The staff of the U.S. Army Corps of Engineers Waterways Experiment Station, who demonstrated their Shell Vibrator System in Texas for the project staff, and furnished information regarding the design of that equipment.

C. B. Thames, District Engineer, District 17, Texas Highway Department, and his maintenance forces who under the direction of R. H. Schleider, Jr., constructed the Pavement Test Facility.

Mr. T. W. Stallworth, who, as Assistant Project Supervisor until the spring of 1965, contributed to the design and supervised the early construction stages of the Pavement Test Facility.

Professor A. M. Gaddis who designed and constructed much of the electronic instrumentation used in the research, and supervised the maintenance of all of it.

H. O. Hartley, who guided the staff in the application of sound and effective statistical principles to the design of the Pavement Test Facility.

Professor J. G. Darroch, who wrote a discussion of the experiment design used in the Pavement Test Facility and offered other valued assistance in the preparation of Chapter Ten.

W. A. Dunlap, who supervised the design-sampling and testing for the Pavement Test Facility.

B. H. Atwell and P. G. Manke, who wrote a discussion, included in Appendix C, of the use of seismic techniques in evaluating flexible pavement materials and made the wave velocity measurements included there.

STANDARD MEASUREMENTS FOR SATELLITE ROAD TEST PROGRAM

SUMMARY

Following completion of the AASHO Road Test in 1960 at Ottawa, Ill., and publication of the research findings in 1962 by the Highway Research Board, it became apparent that further research on highway pavements located throughout the country would be necessary in order to extend the findings to materials and environmental conditions different from those existing at the Road Test. By 1962 several states had begun such studies, some using sections of existing pavements and others constructing new sections. It appeared that the maximum benefit from this work would result only if (a) at least some of the sections in each state conformed to a nationwide statistically designed experiment and (b) measurements on these sections were made in accordance with a single plan encompassing all states conducting these studies.

A nationwide statistically designed experiment was outlined in 1964 in the report issuing from NCHRP Project 1-1 (*NCHRP Report 2A*). It was the general objective of NCHRP Project 1-6, reported herein, to define the variables to be measured on the sections required by the experiment, as well as the equipment and techniques to be employed, the personnel required, and the estimated annual budget for the operation and support of a measurements team.

The principal findings and recommendations are as follows:

1. Two variables are recommended for periodic measurement by nondestructive means—the serviceability index, and the surface deflection caused by a standard loading.

2. Two devices are recommended for determining the serviceability index—the CHLOE profilometer and (as a check) the U. S. Bureau of Public Roads Roughometer.

3. The Dynaflect—a light, trailer-mounted, pavement research tool developed in 1964-65—is recommended for the measurement of surface deflection (or the composite stiffness of the pavement structure).

4. An extensive sampling and testing program aimed at determining the laboratory properties and in situ condition of the materials in each section is recommended for the first year's operation of a national project. The services of a central testing laboratory would be required by this program.

5. Personnel requirement for operating and supporting one measurements team for one year is estimated at 6.25 man-years. It is estimated that this team could process approximately 175 test sections the first year, and approximately 525 test sections annually after the first year.

6. The estimated annual budget for the operation and support of one measurements team, and the average cost per section, are, respectively, $312,500 and $1,786 for the first year and $114,900 and $220 thereafter.

7. Deflections were used in a large satellite project in Texas to determine, at least approximately, the effect of local environment on composite pavement stiffness, and should be used in a national project for the same purpose.

CHAPTER ONE

INTRODUCTION AND RESEARCH PLAN

Prior to the selection of an agency to perform the research reported herein, the Highway Research Board broadly outlined the problems to be attacked, and the research objectives. The outline, in part, was as follows:

> Research under way in NCHRP Project 1-1 points to the necessity of establishing measurement teams equipped, staffed, and trained to make common denominator measurements on the projects in the satellite research program and to insure continuity of these measurements during the life of the program. To date, no mechanism has been established for the formulation and operation of such teams. It is assumed, however, that such a mechanism will be established in the near future, and when this is done, it is desirable that the teams be formed and placed in operation with a minimum of delay.
>
> The measurement program that is considered minimal for the nationwide coordinated satellite research program are outlined in the guidelines prepared under NCHRP Project 1-1 (*NCHRP Reports 2* and *2A*). However, the guidelines do not specify actual items of test equipment, nor do they attempt to define the testing program for the measurement teams insofar as frequency of visits to individual projects and schedules of measurements within projects are concerned.
>
> The objectives of this project, therefore, are as follows:
>
> Phase I: Within the general requirements for common denominator measurements stated in *NCHRP Reports 2* and *2A*, prepare specifications for all personnel, equipment and procedures for the measurement teams, including estimates of cost.
>
> Phase II: Upon approval of the report on Phase I, purchase equipment, fit out a test van, and employ and train the specified personnel for one measurement team. . . .
>
> Phase III: Upon approval of Phase II, schedule and operate the team on five existing satellite test locations to be designated within the United States by the Program Director. . . .

A summary of the research plan prepared by the Texas Transportation Institute and accepted by the Board is as follows:

> *Phase I:* It is assumed that the primary functions of the measurement team are threefold: (1) to sample the materials for testing at a central laboratory, (2) to perform nondestructive tests on the materials in place, and (3) to determine the serviceability index.
>
> Accordingly, in Phase I a study will be made to determine which of the more commonly used laboratory and field tests would be appropriate for this project. A second investigation will evaluate more recent and less widely known laboratory strength tests, including repetitive load tests. A third study will be aimed at evaluating the effectiveness of existing nondestructive methods for estimating the stiffness of individual pavement components, as well as the total pavement structure. . . .
>
> *Phase II:* During Phase II, equipment will be purchased, a test van (or vans) fitted out, and a measurement team employed and trained. Also during this phase, a preliminary Field Testing Manual will be prepared for the use of the measurement crew.
>
> At the end of Phase II, a one-day demonstration of crew, equipment and procedures will be made near College Station, Tex., before the Program Director of the NCHRP.
>
> *Phase III:* During Phase III the measurement team and equipment will be sent to five locations in the United States designated by the Program Director, for the purpose of "shaking down" the crew and equipment and determining if any changes in procedures or apparatus should be made. . . .
>
> In addition to following the procedures outlined in the preliminary draft of the Field Testing Manual, the measurement crew will also conduct special experiments designed to evaluate the measurement error associated with the equipment and testing procedures, as well as the error resulting from normal variations within an average test section.
>
> At the conclusion of the Phase III testing program, the final draft of the Field Testing Manual will be prepared and will be included in the final report of the project. The final report will be intended to provide a basis for equipping, staffing, training and operating several measurement teams. . . .

From the problem statement, the project objectives, and the research plan summarized in the foregoing, it is clear that this research was intended to be one link in a chain of four projects all directed toward the ultimate objective of providing a widely applicable pavement design method based on the results of experiments conducted over a long period of time on test sections located throughout the country. The four projects, the approximate completion dates of the first three, and the titles of the pertinent reports from the first three, are given in Table 1. The fourth project, the National Satellite Road Test Program, has begun (in the sense that a number of local projects are under way), but the concept of "common denominator" measurements to be made on test sections comprising a nationwide, statistically designed study has not yet—to the knowledge of the writers—been executed.

It is the purpose of this report to provide a basis upon which the last stage in the planning of a nationwide experiment can begin.

VARIABLES TO BE MEASURED AND EQUIPMENT RECOMMENDED

The scope of this project was defined in the previous section. This section outlines the recommended scope of the work to be performed by the measurements team in the National Satellite Road Test Program, as it is influenced by the variables selected for measurement and the equipment required to make the measurements.

Variables (General)

A general description of the variables considered to be relevant to the National Satellite Road Test Program was given in Chapter One of the "Guidelines" (*1*) issuing from

NCHRP Project 1-1, and is summarized in Table 2. Table 2 also indicates those variables whose evaluation logically would involve the measurements team, whereas the remaining variables would be evaluated by others. Thus, the team would be concerned to some degree with the measurement of three structural variables, none of the load variables, two climatic or regional variables, and those performance variables that can be evaluated from measurements of deterioration and plastic deformation of the pavement surface.

Table 3, also taken from the "Guidelines" (*1*), summarizes both the variables and the symbols representing them. This table also includes an indication of the involvement of the measurements team.

To clarify the meaning of the symbols representing structural variables, there has been reproduced from the "Guidelines" the sketch of a typical test section (Fig. 1) and reiteration that "other design variables, c_1, c_2, c_3, etc.," do not concern the measurements team.

Finally, by a process of elimination, Table 4 lists only those variables that involve the measurements team and are therefore subject to further study in this research. The following paragraphs discuss each variable listed in Table 4, and the equipment recommended for evaluating it.

Present Serviceability Index

The serviceability index, as described in research reports from the AASHO Road Test, is an estimate of the public's opinion of the relative quality of pavement surfaces, and depends on the first three measurements listed in Table 5.

Research carried out by the Texas Transportation Institute on the Texas Satellite Road Test resulted in a modification of the formula for the serviceability index to include a term for surface texture (*2*), the fourth item listed in Table 5. The Texas research showed that the modified formula gave a better estimate of subjective rating of pavements that include rough textured surfacing materials (surface treatments) than the original formula. Therefore, the use of the modified formula (given in Appendix E) is recommended for the National Satellite Road Test Program.

Because of the importance of the serviceability index, and the ready availability of two devices for measuring its most important component, slope variance, it is recommended that both devices be employed by the measurements team. These devices are the CHLOE Profilometer and the Bureau of Public Roads Roughometer.

A CHLOE Profilometer has been used by the Texas Transportation Institute since the summer of 1962. Although mechanically rugged, it requires frequent calibration, demands the occasional care of a qualified electronics engineer, and must be operated by a well-trained crew. It has the additional disadvantages of a slow operating speed, thus requiring protection from traffic on the test section, and of needing a special means of transporting it between test sections (*3*). Nevertheless, it has furnished data in the quantities and of the accuracy required by the writers in conducting a large satellite road test project, involving hundreds of test sections, for the Texas Highway Department and the Bureau of Public Roads. Further details concerning the CHLOE Profilometer and associated instruments are given in Chapter Two.

The Bureau of Public Roads Roughometer has the advantage over the CHLOE Profilometer of a higher operating speed (20 mph) and one-man operation. It has the disadvantage that its output (inches per mile) is inaccurate for test sections less than about 2,000 ft in length. Further

TABLE 1

STAGES IN PLANNING FOR NATIONWIDE EXPERIMENT ON ROAD TEST EXTENSION

PROJECT	REPORT
AASHO Road Test	*HRB Spec. Rep. 61E*, "The AASHO Road Test, Pavement Research" (1962).
NCHRP Project 1-1	*NCHRP Report 2A*, "Guidelines for Satellite Studies of Pavement Performance" (1964).
NCHRP Project 1-6	*NCHRP Report 59*, "Standard Measurements for Satellite Road Test Program" (1968).
National Satellite Road Test Program	—

TABLE 2

GENERAL BASIS FOR STUDIES OF ONE PAVEMENT TYPE [a]

TYPE OF VARIABLE	MEASUREMENT	MEAS. TEAM IN-VOLVED
Design variables		
Structural:		
Pavement structure:	Strength characteristics	Yes
Surface courses	Thicknesses of courses	Yes
Base courses (if any)	Other design features	No
Subbase courses (if any)	Composite strength	Yes
Roadbed material		
Load	Accumulated axle loads	No
	Years of service	No
	Rate of accumulation	No
Climatic and regional	Conditions of precip., moisture, temp., and frost	No
	Topography	No
	Relative strength in different climates	Yes
	Regional factors	Yes
Performance variables		
Surface behavior	Deformation and deterioration	Yes
	Present serviceability	Yes
	Performance	

[a] From Ref. *1*, p. 3a.

TABLE 3

GENERAL BASIS FOR RELATIONSHIPS AMONG VARIABLES FOR ONE PAVEMENT TYPE [a]

TYPE OF VARIABLE	SYMBOLS [b]	MEAS. TEAM INVOLVED
Performance variables		
Meas. of surface deformation and deterioration	x_1, x_2, x_3, etc.	Yes
Present serviceability index	$p(x_1, x_2, x_3$, etc.)	Yes
Performance index	$P(p_0, \Sigma L^c$ when $p = 2.5)$	Yes
Design variables		
Structural:		
Thicknesses	h_1, h_2, h_3	Yes
Strength characteristics	s_1, s_2, s_3, s_4	Yes
Other design features	c_1, c_2, c_3, etc.	No
Composite strength	S	Yes
Load and time	ΣL, ADL, Y	No
Climatic or regional:		
Climatic variables	$v_1, v_2, v_3, \ldots$	No
Relative strength	RS	Yes
Regional factor	RF	Yes

[a] From Ref. *1*, p. 15a. [b] Variables outside ( ) defined by those inside. [c] ΣL does not involve measurements team.

TABLE 4

SUMMARY OF VARIABLES INVOLVING THE MEASUREMENTS TEAM

TYPE OF VARIABLE	SYMBOLS [a]	FREQUENCY OF MEAS. [b]
Performance variables		
Meas. of surface deformation and deterioration	x_1, x_2, x_3, etc.	R
Present serviceability index	$p(x_1, x_2, x_3$, etc.)	R
Performance index-time series data	p_0(init. serv. index)	1
Design variables		
Structural:		
Layer thickness	h_1, h_2, h_3	1
Layer strength	s_1, s_2, s_3, s_4	1
Composite strength	S	R
Climatic or regional:		
Relative strength	RS	R
Regional factor	RF	R

[a] Variables outside ( ) defined by those inside. [b] 1 = one-time measurement; R = recurring measurement.

details of the roughometer, including the description of a method for converting its output to slope variance, are given in Chapter Three.

Layer Thickness

The thicknesses of layers shown on highway construction plans sometimes do not equal actual thicknesses. Therefore, $h_1, h_2, \ldots$ should be measured on the completed section by the measurements team with the assistance of local state highway department forces. Recommended procedures are given in Chapter Seven.

Layer Strength and Composite Strength (Flexible Pavements)

The variables, $s_1, s_2, \ldots$ listed in Table 4 were frequently referred to in the "Guidelines" (*1*), but were never clearly defined. This was by intention, and as a result it became an objective of the present research to provide a definition of—and to recommend the means for estimating—the strength of the materials composing each layer in a flexible pavement.

Accordingly, laboratory strength, S, of a flexible pavement material is defined here as the ultimate compressive strength of a 6-in. diameter and 8-in. high cylindrical specimen of the material molded in accordance with standard Texas Highway Department procedures (*4*), and tested at a lateral pressure of 5 psi (see Appendix A for procedure). Also, the field compression coefficient, F, for a flexible pavement material is defined as a measure of the amount that the material in situ tends to compress and rebound when subjected to compressive forces set up by wheel-loads, under the stipulation that F is independent of the thickness and the position of the layer in which the material occurs. It is further postulated that F is a function of S, and also of the regional factor listed in Table 4.

The definitions of S and F, and their postulated interdependence, were considered to be hypotheses that had to be

TABLE 5

MEASUREMENT DESCRIPTIONS, SYMBOLS, AND METHODS OF OBTAINMENT

MEASUREMENT NUMBER	DESCRIPTION	SYMBOL	METHOD OF MEASUREMENT
1	Slope variance in wheel paths	$\overline{\text{SV}}$	AASHO Road Test Profilometer or CHLOE Profilometer
2	Amount of cracking and patching	C + P	Estimated visually or measured with tape
3	Rut depth (flexible pavements only)	$\overline{\text{RD}}$	Rut-depth indicator
4	Surface texture (flexible pavements only)	T	Texturemeter

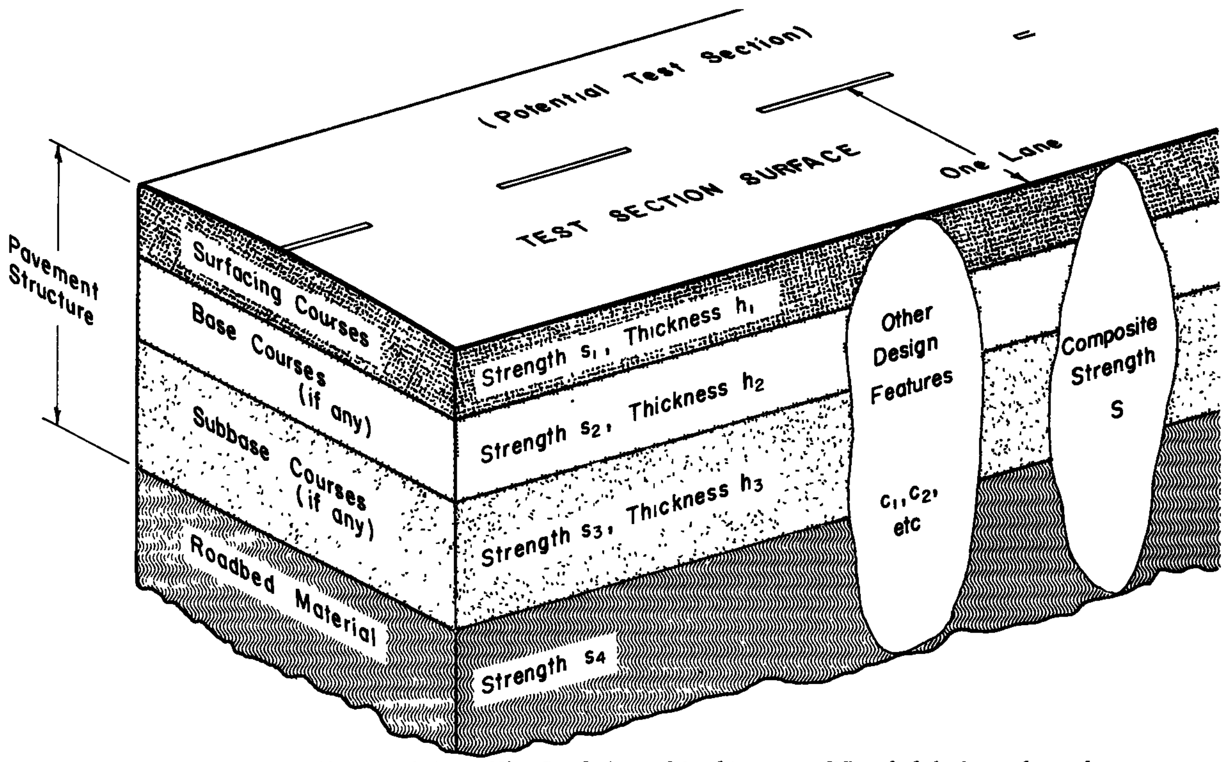


Figure 1. Typical test section (from Ref. 1, p. 60). Symbols used in this report differ slightly from those shown.

tested in this research to the extent that time and funds permitted. Because *F* was by definition an in situ value, it was necessary to evaluate it by nondestructive means on full-scale test sections. And because by definition *F* was independent of the thickness and position of the layer in which the material occurred, it was necessary to use sections in which *F* was correlated with neither thickness nor position of layers, a requirement that immediately eliminated from consideration test sections on existing highways. (Regular highways could not be used because of the universal practice of building into a highway a correlation of *S*—and therefore *F*—with position; the greater values of *S* always occur in the upper layers.) Thus, the research called for the construction of a statistically designed pavement test facility consisting of a number of full-scale test sections. Such a facility was constructed and is described in Chapter Ten.

Also required was nondestructive testing equipment that might be capable of proving the existence of the variable, *F*, and measuring its value. Two nondestructive testing systems were tried. The first and perhaps better known was the Shell Vibrator system, named after its developer, the Shell Oil Company. The second was the Dynaflect system, manufactured by the Lane-Wells Company of Houston, Tex.

Both systems apply an oscillating load to the pavement. In the Shell system the load frequency can be varied at will, while the amplitude cannot. In the Dynaflect both load amplitude and frequency are fixed. The response measured by the Shell system is the length of the wave traveling in the pavement surface at a known, controlled frequency. The response measured by the Dynaflect system is the maximum amplitude of the vertical motion at a selected point on the pavement caused by fixed loading conditions. The data from the Shell system consist of several pairs of values, one of frequency and the other of wavelength. The data from the Dynaflect also consist of several pairs of values, one of position with respect to the load and the other of maximum amplitude of vertical motion. (Further details of the design and operation of these devices are given in Chapters Four and Five.)

In theory the two devices seemed equally capable of performing the required job. In practice the Dynaflect was found to be preferable in the following respects:

1. The Dynaflect output had been previously correlated (*5*) with static, rebound deflections resulting from a 9,000-lb wheel load. Thus, Dynaflect data could be tied to a great volume of previous and current research involving deflections measured by the Benkelman beam, including measurements made at the AASHO Road Test.*

2. The Dynaflect could be operated at lower cost. One man could collect Dynaflect data sufficient to define the

* The correlation with Benkelmen beam deflections described by Scrivner and Moore (*5*) was accomplished using the original (1964) model of the Dynaflect. The later (1966) model recommended for use by the measurement team was compared with the original model to obtain correlation with Benkelman beam deflections. The following approximate relationship was found:

Rebound deflection resulting from a 9,000-lb wheel load and measured by Benkelmen beam = 22.4 x Dynaflect (1966 model) deflection indicated by the geophone located midway between the load wheels.

deflection basins at 15 stations on a 2,500-ft test section while three men were collecting Shell system data at one station on the same section.

3. From an analysis of Dynaflect data taken on the 27 test sections making up the pavement test facility (see Chapter Ten) it was possible to estimate logical values of *F* for the seven materials appearing in the facility, and to show a functional relationship between *F* and *S*. This was not the case for the Shell system data obtained on the test facility, although analysis of those data and other data gathered on Texas highways is continuing.

It was not possible to estimate logical values of *F*, even from Dynaflect data, taken on single test sections; this job could be done only when the full range of the 27-section experimental design described in Chapter Ten was used. Nevertheless, the fact that it could be done at all indicated that the Dynaflect was responding to the strength, the position, and the thickness of the individual layers. This fact, coupled with the first two advantages mentioned previously, led the writers to recommend the use of the Dynaflect rather than the Shell system to evaluate the composite strength of flexible pavements, the last of the structural variables listed in Table 4. However, the determination of in situ strengths of pavement layers with the Shell system is still under investigation by the Texas Transportation Institute as well as by numerous other research agencies in this country and abroad. A description of this system is included herein (Chapter Five) because it may become desirable later to add it to the measurement team's equipment.

Relative Strength (Flexible Pavements)

The variable relative strength (or relative composite strength) is described in the "Guidelines" (*1*) as a measure of the effect of climate, or of the effect of other factors related to location, or of the effect of a combination of those factors, on the performance of test sections. For example, a flexible pavement subject to cycles of freezing and thawing may deflect widely differing amounts under the same load, depending on the season of the year in which the deflection is measured. The ratio of the deflection taken during the late fall (before freezing sets in) to the deflection taken at any other time is thus a climate-dependent variable that measures the relative strength of the pavement at the time the deflection is determined. On the other hand, two widely separated but initially identical flexible pavements may after a period of time have widely differing deflections averaged over the four seasons. The ratio of these two averages is then a location-dependent variable that measures the regional effect on pavement behavior.

An example of how the existence of a regional effect can be discovered and the regional boundaries delineated by means of deflection ratios is given in Chapter Ten under "Engineering Implications." The data used were gathered on the Texas satellite road test sections and on the special sections comprising the pavement test facility mentioned earlier. The method is described and documented herein for the following reasons:

1. The method may be applicable (with some minor changes) to the National Satellite Road Test Program.

2. The fact that regions were demonstrated to exist appears to be additional evidence of the validity of the analysis of deflections made on the A&M Pavement Test Facility, and of the value of deflection ratios as a measure of relative strength.

3. The great difference between regions observed in Texas lends emphasis to the need for the study of regional effects in the National Satellite Road Test Program.

4. The method appears to be a means for retrieving useful results from a satellite road test years before all the performance data could be gathered and the final analysis made.

Regional Factor (Flexible Pavement)

This variable, according to the "Guidelines" (*1*), expresses the difference between the observed performance of pavements located in a well-defined region and the observed performance of pavements of similar design located at the AASHO Road Test. Evaluation of the regional factor thus would depend on layer strength, layer thickness, and serviceability index data gathered by the measurements team, combined with traffic and load data obtained from other sources. Evaluation of the regional factor depends in large part on the loss in serviceability index and it should be pointed out that it will require several years of observations before it will be possible to detect significant trends in these data.

An example of a regional factor not dependent on performance data, and therefore potentially available much earlier in the life of a satellite road test project, is given in Chapter Ten under "Engineering Implications," referred to previously in the discussion of relative strength. As used in the example, the regional factor expresses the difference between the observed deflection of pavements located on existing highways throughout Texas and the computed deflection of pavements of similar initial design located at the A&M Pavement Test Facility. With minor modifications, it appears that the method described in Chapter Ten could be used to evaluate regional factors based on deflection ratios, for test sections located anywhere in the country, provided *S* data and layer thickness data are available for those sections.

Composite Strength, Relative Strength, Regional Factor (Rigid Pavements)

As indicated previously, it was possible to conduct rather extensive physical research leading to firm recommendations regarding procedures for evaluating composite strength, relative strength, and regional factors for flexible pavements from deflection data. This was possible primarily because a large share of the cost was borne by the Texas satellite road test program, in which the main effort to date has been directed toward flexible pavements.

A comparable research program involving rigid pavements was not possible in this project because of time and funding limitations. Current unpublished research by others,* however, suggests the probability that the Dynaflect can be used to determine the composite and relative strength of rigid pavements. It is therefore recommended that the measurement team determine the average deflection basin on at least five slabs in the slab interior, as well as near edges, near corners, and across transverse joints, on the first visit to each rigid pavement section. If analysis of the resulting data shows that the measured deflections are indeed related to the design variables including the regional variable, the measurement of Dynaflect deflections on rigid pavements should be continued on a routine basis until sufficient data have been accumulated to establish firmly any seasonal and regional effects that may be present.

The identification of regional effects is provided for in the experiment design for rigid pavements recommended in the "Guidelines" (*1*, pp. 40-41). If this or a similar design is not used in the National Satellite Road Test Program, it may be necessary to construct a special rigid pavement experiment with test sections confined to a very small area, such as the flexible pavement experiment described in Chapter Ten, to insure that a relationship not confounded with regional effects can be obtained between Dynaflect deflections, layer thickness, layer position, and layer strength. The relationship could then be used in the manner described in Chapter Ten under "Engineering Implications" to establish regional effects, if they exist.

Layer Strength (*Rigid Pavements*)

In the absence of experimental evidence to the contrary, it is assumed from existing theory that the following properties of portland cement concrete pavement significantly influence its performance, and it is recommended that they be determined in the laboratory:

1. Compressive strength.
2. "Split tensile" strength.
3. Modulus of elasticity.
4. Poisson's ratio.

* An example of current research in the use of Dynaflect on rigid pavements is that being conducted by the Ohio River Division Laboratories of the U. S. Corps of Engineers. Included in the investigation is the use of Dynaflect deflections to determine the stiffness of subgrades, the presence of cracking in the bottom of the slab, and the relative efficiency of load transfer at joints.

TABLE 6

SUMMARY OF VARIABLES AND EQUIPMENT

VARIABLE	RECOMMENDED EQUIPMENT	APPROX. COST ($)
Serviceability index	CHLOE Profilometer	6,900
	USBPR Roughometer	10,200
Layer thickness	Drill rig	—[a]
Layer strength	Sampling and testing	400[b]
Composite strength, relative strength, and regional factor	Dynaflect	25,100
Total		42,600[c]

[a] To be leased from local highway department.
[b] Does not include central laboratory equipment.
[c] Excluding three towing cars.

It appears likely that these four properties can be used eventually to define the variable, S_1. Details of sampling and testing techniques are given in Chapter Seven.

The "laboratory strength" variable recommended for the subbase and subgrade layers is the same as that recommended for flexible pavements; i.e., the ultimate compressive strength at a lateral pressure of 5 psi in accordance with Texas Highway Department standard procedures (*4*).

Summary

Table 6 summarizes the foregoing recommendations regarding the variables to be measured by the team and the equipment to be used. Also shown in the table are the approximate costs for the equipment systems.

It should be pointed out that current research activity in the area of highway pavements will eventually lead to more sophisticated measuring equipment, better measuring techniques, and more positive identification of the variables that affect pavement life. The recommendations made herein are based on the present state of the art, and should be checked against the latest developments before they are implemented.

CHAPTER TWO

CHLOE PROFILOMETER SYSTEM

The CHLOE Profilometer system is designed to measure variations in the longitudinal and transverse profile of a highway pavement surface. The system consists of (1) the CHLOE Profilometer (including a special trailer to transport it), (2) the towing vehicle, (3) the rut depth gauge, and (4) the Texturemeter. Figure 2 shows the units which comprise the system.

The profilometer (Figs. 3 and 4) is a unit of specialized research equipment developed at the AASHO Road Test and later manufactured in quantity under the sponsorship

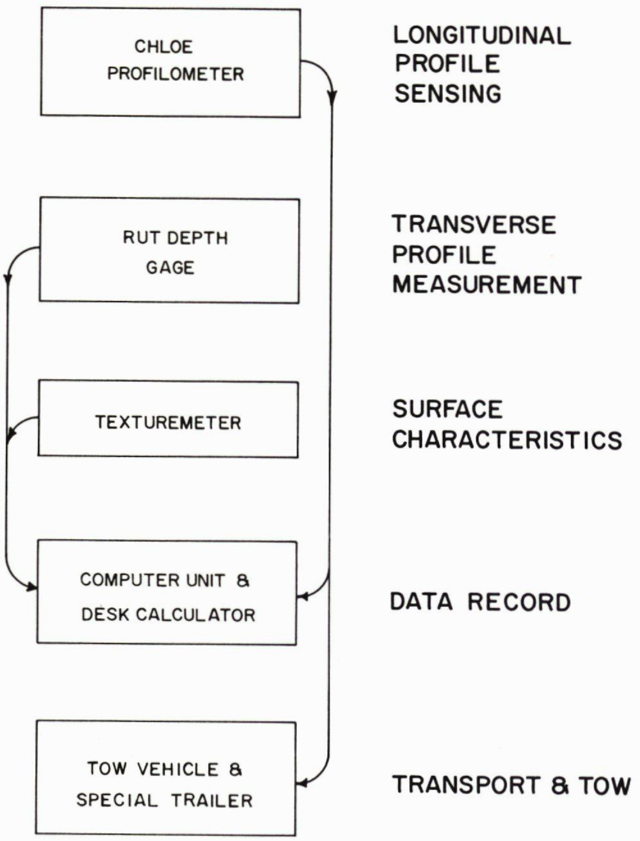


Figure 2. The CHLOE Profilometer system and data flow chart.

Figure 3. The CHLOE Profilometer in operation.

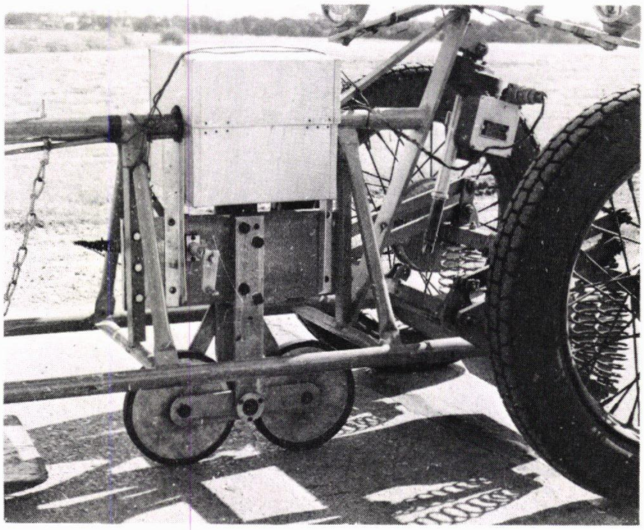


Figure 4. CHLOE Profilometer, close-up of slope wheels.

Figure 5. The rut depth gauge affords a measure of the transverse profile of the pavement.

of the U. S. Bureau of Public Roads. The profilometer, including the rut depth gauge (Fig. 5), may be obtained from that agency at a cost of $5,200. This cost does not include two heavy-duty storage batteries required to power the unit. Specifications for the profilometer and rut depth gauge are available from the Bureau of Public Roads.

The trailer for the profilometer is shown in Figure 6 and discussed in detail in a report available on request (*3*). The trailer can be custom built at a cost of approximately $250 from plans available on request from the Texas Transportation Institute.

The towing vehicle should be wired so that the vehicle generator is used to charge profilometer batteries and it should be equipped with an AC-DC converter to operate a small automatic calculator. The vehicle should also be equipped with three trailer hitches—one at the center of the vehicle and one over each wheel path. The vehicle modifications, including a flasher signal for the roof of the vehicle to caution traffic during tests, can be accomplished at a cost of approximately $250.

The Texturemeter (Fig. 7) was developed by the Texas

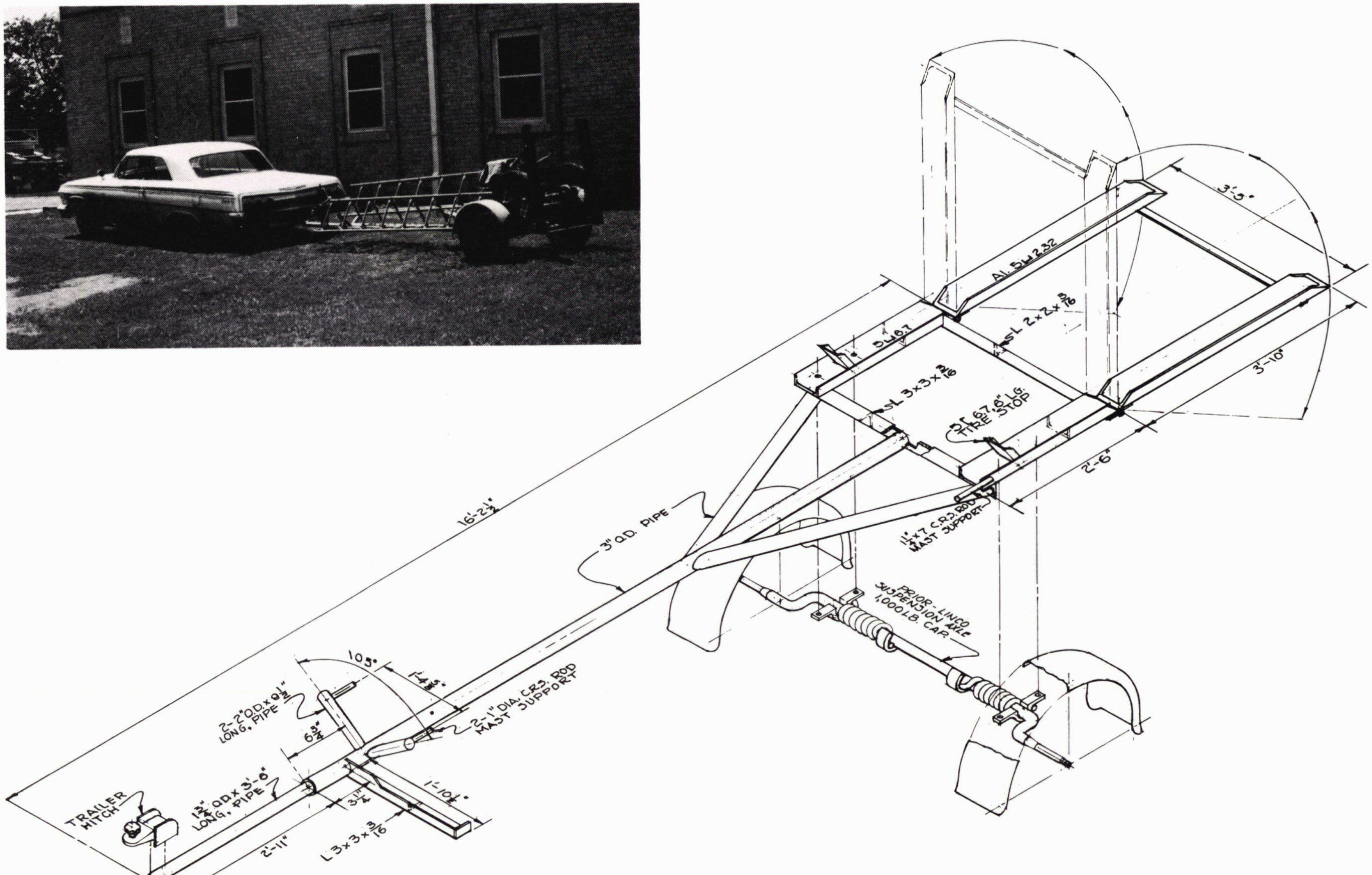


Figure 6. Trailer details for CHLOE Profilometer.

Figure 7. The Texturemeter applied to a laboratory specimen as asphaltic concrete. Road surfaces give dial readings ranging from 0 to about 0.100 in.

Transportation Institute in connection with another research project and is discussed in detail in a report available on request (*2*). The device can be custom built at a cost of approximately $500 from plans available from the Texas Transportation Institute.

The total cost of the CHLOE Profilometer system is approximately $6,900, which includes the following:

CHLOE Profilometer and batteries	$5,250
Profilometer trailer	250
Towing vehicle modifications	250
Texturemeter	500
Calculator (Friden Model CW-10)	600
Volt-ohm milliammeter (for trouble shooting)	50

CHAPTER THREE

USBPR ROUGHOMETER SYSTEM

Inasmuch as both the CHLOE Profilometer (described in Chapter Two) and the Bureau of Public Roads Roughometer have been extensively used for determining road roughness, it is recommended that the measurements team employ both instruments, at least in the initial phases of the National Satellite Road Test Program. With this in view, an extensive correlation study of the two instruments was made on Texas highways.

DESCRIPTION OF EQUIPMENT

The Roughometer used on this project, borrowed from the Bureau of Public Roads, is shown in Figure 8. The system consists of the Roughometer unit and a specially equipped tow vehicle.

The Roughometer unit was developed and constructed by the Physical Research Division of the U. S. Bureau of Public Roads. Such units as are required for the measurements team could be (1) obtained from the Bureau of Public Roads, (2) built from plans furnished by the agency, or (3) purchased from commercial sources.* It is recommended, however, that if several teams are equipped with the Roughometer, all units be obtained from the same source. The approximate cost of one unit is $10,000.

Tow vehicle modifications, costing approximately $100, include installation of (1) a special towing connection, (2) electrical connectors and electrical conductor cables leading from the rear of the vehicle to the operator's seat, and (3) a flasher signal for the roof of the vehicle to caution traffic during tests. The exact details of the modifications

* Possible sources are: Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.; and TEST Lab Corp., 564 W. Monroe St., Chicago 6, Ill.

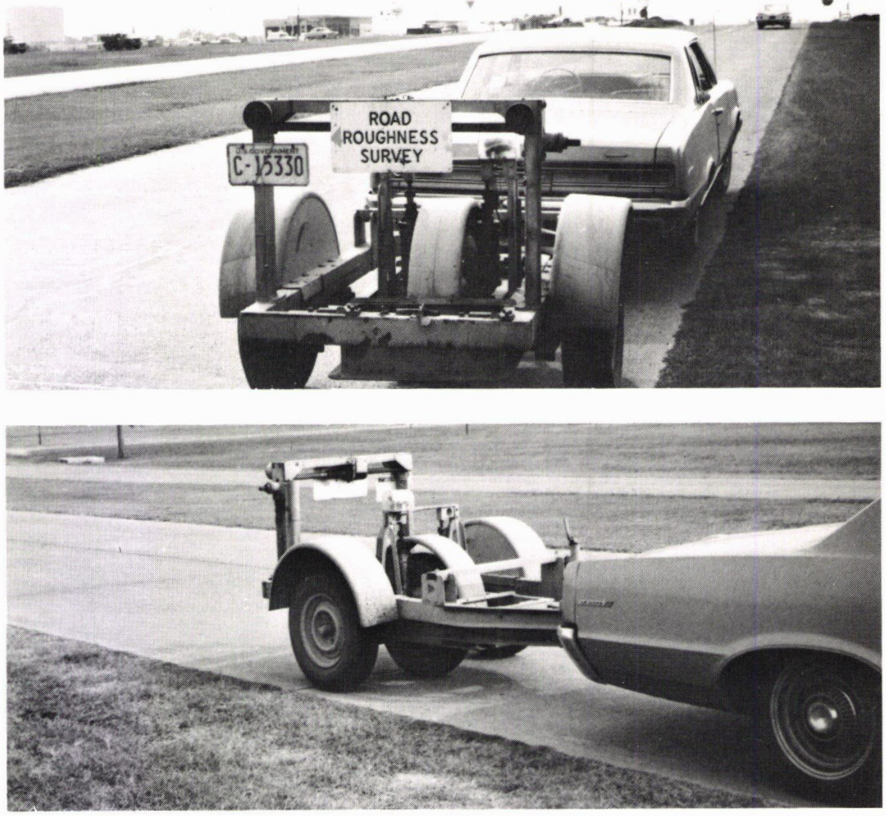


Figure 8. BPR Roughometer.

to the towing vehicle vary with the type of Roughometer acquired.

The Roughometer output is displayed on a counter board consisting of two electric counters and a switch. By adding an additional switch and an additional pair of counters (Fig. 9), multiple sections can be tested without stopping. That is, data from one counter bank can be recorded while counting on the second counter bank. A wiring diagram of a counter board with two counter banks is shown in Figure 10. This modification can be made in the counter board supplied with the unit at an approximate cost of $100.

CORRELATION OF ROUGHOMETER AND CHLOE PROFILOMETER

In connection with another research project, the Texas Transportation Institute made measurements of slope variance (using the CHLOE Profilometer) and surface texture (using the Texturemeter) on several hundred flexible pavements on Texas highways in 1964-5. The slope variance and texture recorded was the average for both wheel paths in each test section.

During the same period, as a part of the project reported herein, the BPR Roughometer was used to measure, in the outer wheel path only, the roughness (inches per mile) of 213 of the test sections. The Roughometer data were compared with the data taken with the CHLOE Profilometer system for the purpose of establishing the degree of correlation between the two systems.

Several mathematical models relating the output of the three instruments—CHLOE Profilometer, Roughometer, and Texturemeter—were investigated. The mathematical model selected was the following (logs are to the base 10):

$$\log\,(1 + \overline{\mathrm{SV}}) = A_0 + A_1 \log R + A_2 \log\,(1 + T) + \text{error} \tag{1}$$

in which

$\overline{\mathrm{SV}}$ = slope variance (average of both wheel paths) measured by CHLOE Profilometer;

R = Roughometer output (inches per mile); and

T = average surface texture, measured with the Texturemeter.

Using standard regression techniques, the values of the constants were found to be $A_0 = -3.166$, $A_1 = 1.995$, and $A_2 = 0.236$.

The standard deviation in the dependent variable, log $(1 + \overline{\mathrm{SV}})$, was 0.152, and the squared correlation coefficient was 0.80. Thus, 80 percent of the variation in log $(1 + \overline{\mathrm{SV}})$ was explained by Roughometer and Texturemeter data, and two-thirds of the predictions made from Eq. 1 had an error of 0.152 or less.

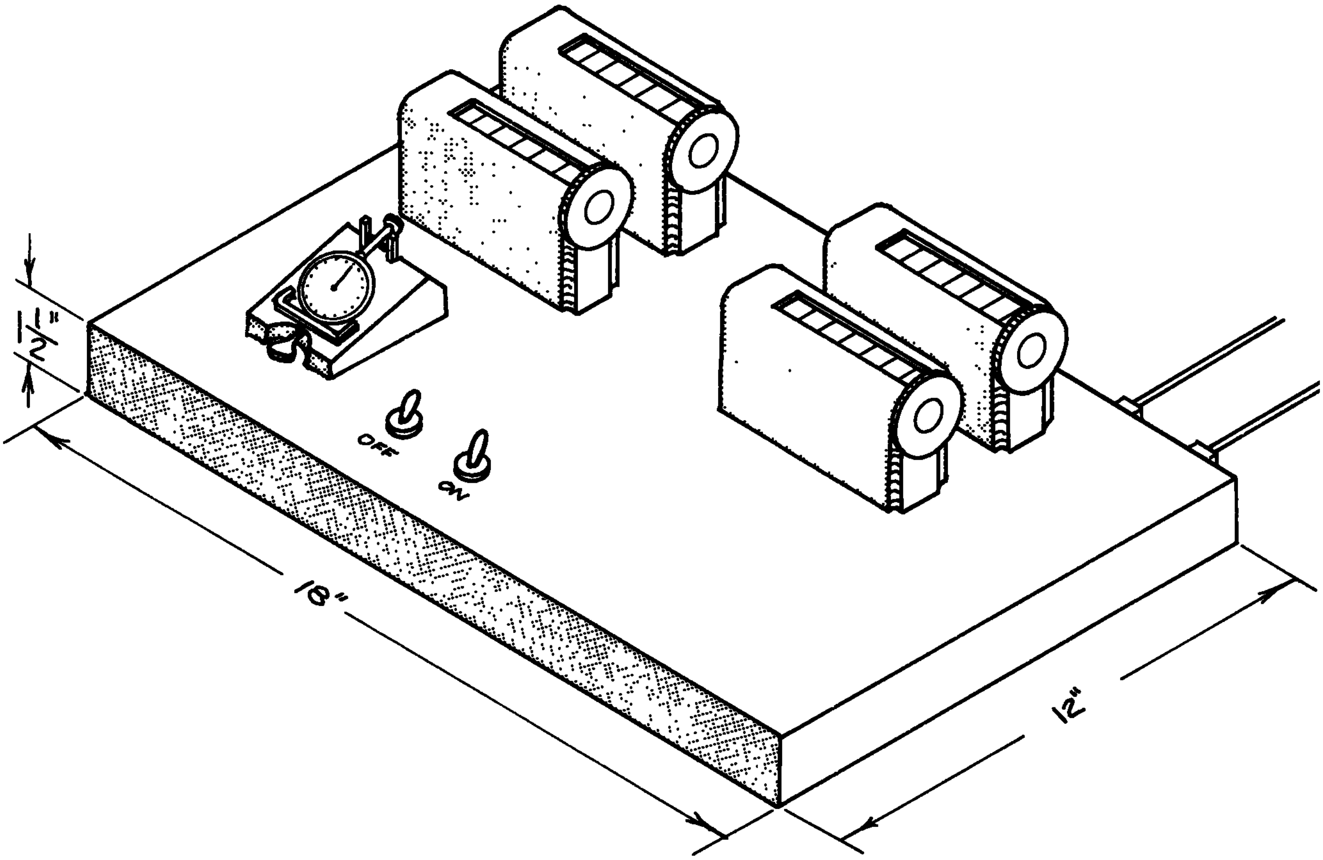


Figure 9. Roughometer counter board.

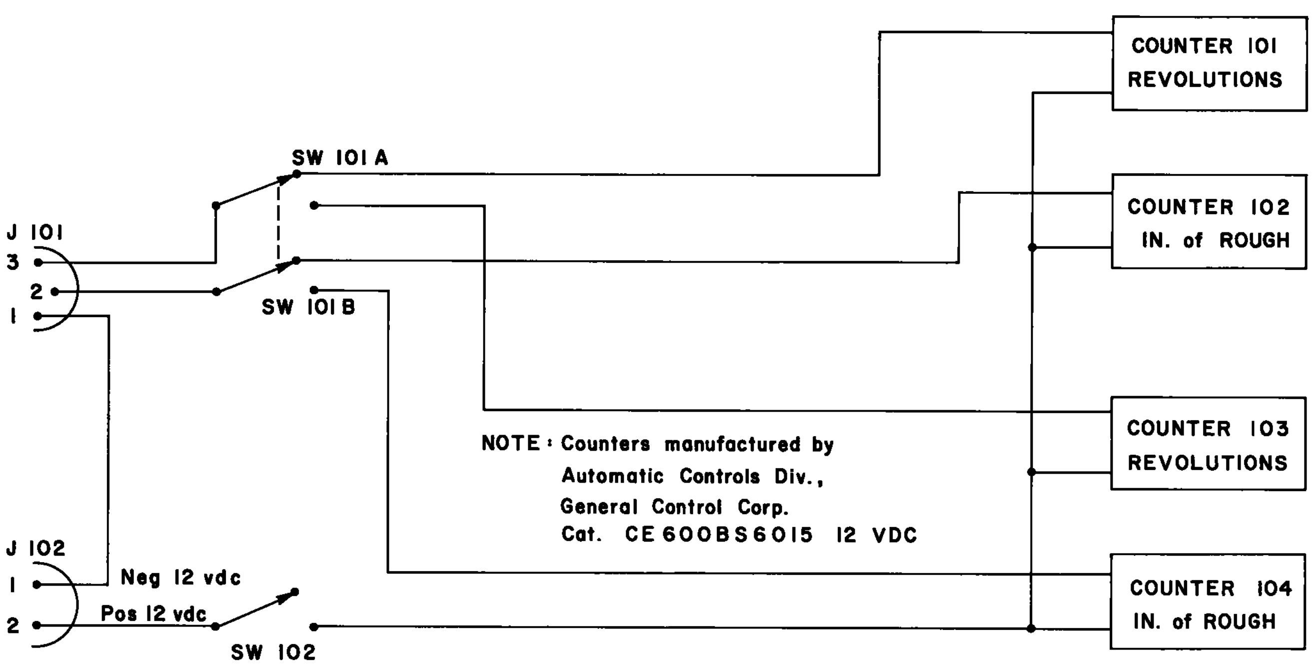


Figure 10. Wiring diagram for Roughometer counter board.

With the texture term omitted the model becomes

$$\log (1 + \overline{SV}) = A_0 + A_1 \log R + \text{error} \quad (2)$$

For this case the constants were $A_0 = -3.503$ and $A_1 = 2.254$.

The standard deviation was 0.186, and the squared correlation coefficient was 0.70. Thus, 70 percent of the variation in log $(1 + \overline{SV})$ was explained by Roughometer data alone, and two-thirds of the predictions made from Eq. 2 had an error of 0.186 or less.

Eq. 1 may be used for estimating from Roughometer and Texturemeter data the value of the slope variance term in the serviceability index equations given in Appendix E (Eqs. E-1 and E-2).

CHAPTER FOUR

LANE-WELLS DYNAFLECT

The Lane-Wells Dynaflect is a one-man-operated device (Fig. 11) that induces and measures the deflection of the roadway surface. The device is rugged, rapid, reliable, and more economical to operate than any other similar equipment known to the writers. It is mounted on a small two-wheel trailer that can be towed at normal highway speeds by a passenger automobile and stopped briefly at a test location to make deflection measurements. The operator of the Dynaflect also serves as the driver of the towing vehicle.

Roadway deflections determined with the Dynaflect on flexible pavements have been shown to be well correlated with static deflections measured by conventional means—that is, the rebound deflection of a 9,000-lb wheel load measured with a Benkelman beam (*5*). This correlation is considered good evidence that the Dynaflect responds to much the same properties of a flexible pavement that govern the deflection produced by a wheel load.

The two basic components of the Dynaflect are the dynamic force generator that loads the roadway surface and the deflection measuring system. The dynamic force generator consists of a pair of counter-rotating eccentric masses arranged so that the cyclic force produced is transmitted to the roadway through a pair of steel wheels. The static weight of the trailer exceeds the dynamic force produced by the rotating weights. Consequently, there is no tendency for the device to lose contact with the pavement. The dynamic force generator produces steady-state cyclic vertical displacement in the vicinity of the load wheels. Conventional geophones (seismometers) are used to measure the vertical motion of the pavement surface. Because the displacement is cyclic, a sensor does not require a fixed reference outside of the deflection basin. Geophones can be placed at any desired location; thus, the magnitude of the vertical motion at any point, or at several points in the deflection basin, can be determined. The control unit for the current model of the Dynaflect is designed to accommodate six sensors at one time (Figure 12).

The Dynaflect may be purchased for approximately $10,000 from the Lane-Wells Company, Houston, Tex. Purchase of the unit also provides a detailed operator's manual that contains instructions for the installation of a trailer hitch and control unit wiring. This installation in the towing vehicle, plus a flasher signal for the roof of the vehicle to caution traffic during tests, can be accomplished at a cost of approximately $100.

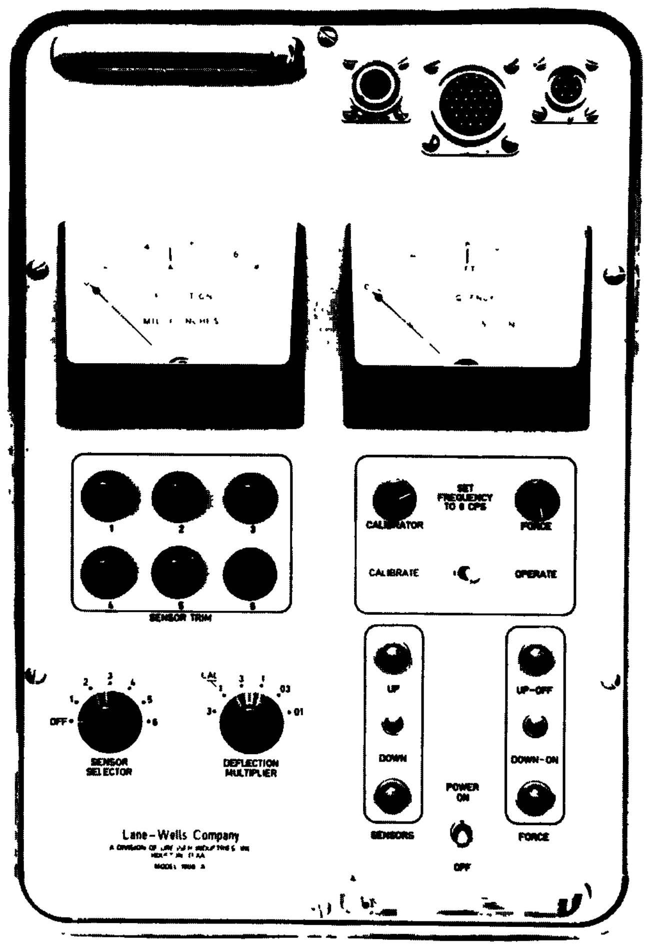


Figure 12. Lane-Wells Dynaflect operation control panel.

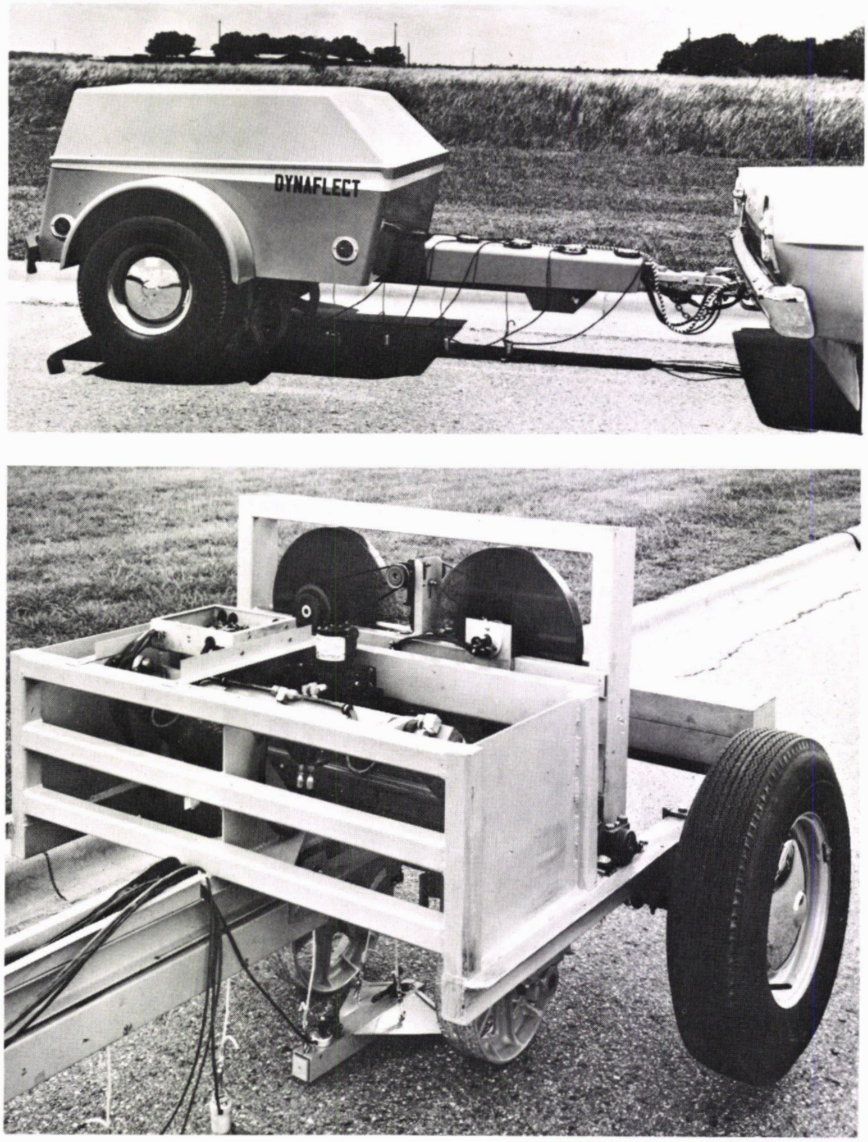


Figure 11. Lane-Wells Dynaflect as used in tests (upper) and with trailer body removed (lower).

CHAPTER FIVE

SHELL VIBRATOR SYSTEM

The Shell Vibrator system, developed by the Shell Oil Company in Amsterdam, measures the length of the ground wave induced by a vibrator operating at a known frequency. The system described herein was patterned after a similar device constructed by the U. S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss.

In operation on a test section the frequency is varied, in steps, in the range from about 40 cps to about 2,000 cps, and the wavelength corresponding to each selected frequency is measured. Thus, the data consist of a list of frequencies and the corresponding wavelengths measured on the surface of the pavement.

Although the Shell system is not recommended for use by the measurements team at this time, it is still being intensively studied by the Texas Transportation Institute in connection with another continuing research project. If favorable results are obtained in the future, a complete report will be made available to the Highway Research Board, subject to the approval of the sponsors.

The system consists of five major functional components. They are: (1) a portable power source, (2) the test equipment, (3) an environmental unit, (4) a communications unit, and (5) the transportation unit. The relationship of these units is shown in Figure 13. The test equipment is diagrammed in more detail in Figure 14 and major components are shown in Figure 15.

The total cost of the Shell Vibrator system is approximately $13,000, which includes the following:

Portable power source	$1,700
Test equipment	6,450
Air conditioner (Friedrich Model 34242)	450
Communications unit	150
Truck van	3,500
Van modifications (table, air cond. ducts, etc.)	750

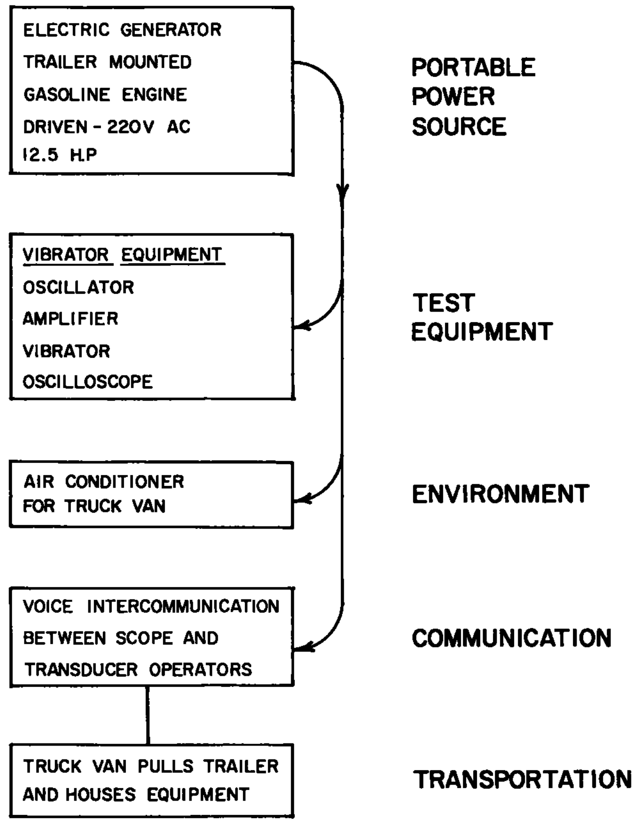


Figure 13. Relationship of units of the Shell Vibrator system.

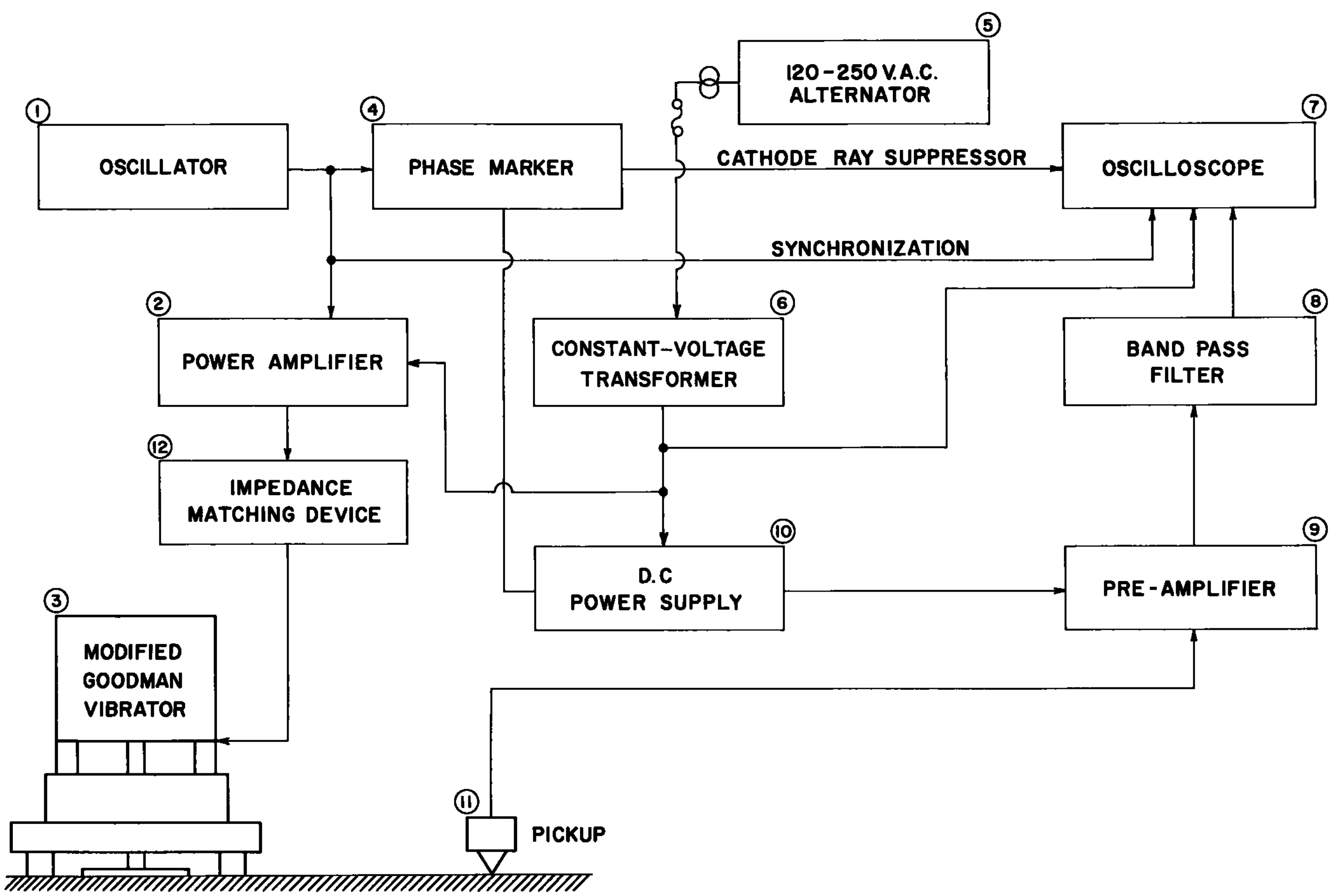
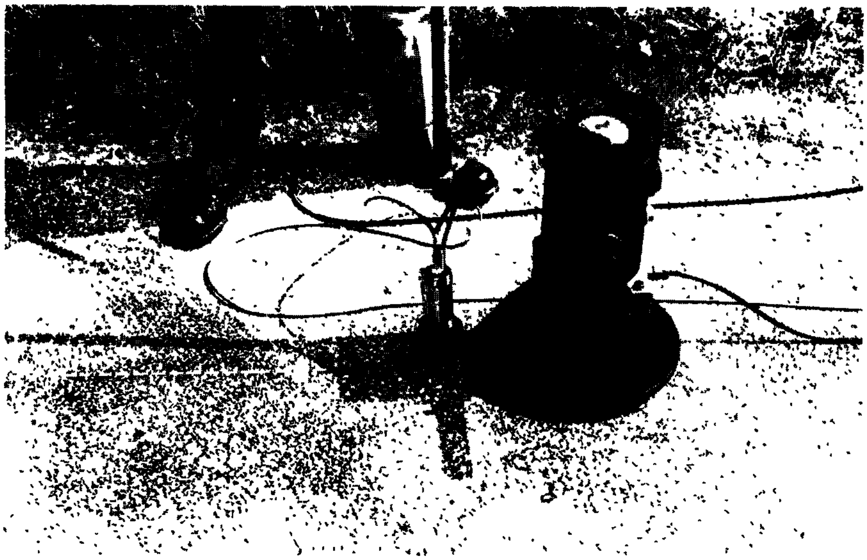


Figure 14. Schematic of Shell Vibrator system test equipment.



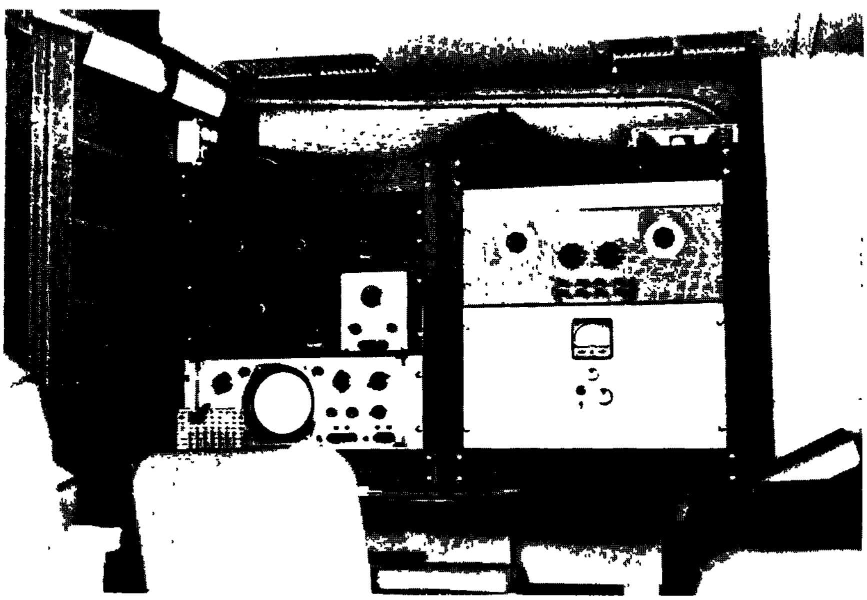


Figure 15. Shell Vibrator system (upper) vibrator generator and transducer units and (lower) test equipment installed in van.

CHAPTER SIX

PERSONNEL REQUIREMENTS AND JOB DESCRIPTIONS

Personnel qualifications and job descriptions for support personnel (exclusive of laboratory personnel) and measurements team personnel are given in this chapter. The qualifications stated are those presently used by the Pavement Design Department, Texas Transportation Institute. The list of personnel (Table 7) gives the approximate time required for each member.

MEASUREMENTS TEAM SUPPORT PERSONNEL

1. *Project Supervisor*

Qualifications: College graduate; registered professional engineer; a minimum of five years research experience; a thorough knowledge of statistics, theory of vibration, pavement design, and soil mechanics; and a basic knowledge of electronic data processing capabilities.

Job description: The project supervisor will interview and approve the selection of personnel, direct the research and report writing, supervise the analysis of all field data, formulate all plans of operation, and coordinate the activities of the measurements team with testing schedules.

2. *Assistant Research Engineer*

Qualifications: College graduate in civil engineering; experienced in data processing, electronic computer programming, and personnel management; have a thorough knowledge of pavement design and soil mechanics.

Job description: The assistant research engineer will assist in the supervision of the project employees, direct the processing of the field data, design data forms to minimize the handling of field data; assist the project supervisor in the preparation of data for analysis; prepare progress reports and assist in preparing and writing technical reports. He is responsible to the project supervisor.

3. *Research Assistant*

Qualifications: College graduate; specialized training in computer science; experienced in electronic computer programming and data processing; age 20 to 40; a basic knowledge of mathematics, physics, and statistics desirable.

Job description: Under the direction of the project supervisor, the research assistant programs all research problems, schedules work sequence, retrieves information, and assists in the analysis of the programs. He processes all field data. He also assists the assistant research engineer in the design of data forms which will facilitate the gathering and processing of all field data.

4. *Electronics Systems Engineer*

Qualifications: College graduate in mechanical or electrical engineering, with a minimum of four years experience in nondestructive testing; a thorough knowledge of vibration characteristics, stress-strain transducers, and electronic recording systems; vibrations phenomena research experience preferred.

Job description: The electronic systems engineer directs work of technicians in building and troubleshooting all nondestructive test and electronic recording equipment; designs new test equipment; modifies present equipment when necessary. His modification and installation schedule is to be arranged with the measurements team field supervisor. He is responsible to the project supervisor.

5. *Technician II (Electronics)*

Qualifications: High school graduate, age 20 to 40, four years experience in electronics, radio and television repair; above average intelligence with a basic knowledge of electronic systems design.

Job description: The technician II (electronics) will assist the electronic systems engineer in design, modification, assembly, and maintenance of all electronic and mechanical equipment.

TABLE 7

LIST OF PERSONNEL FOR OPERATING AND SUPPORTING ONE MEASUREMENTS TEAM

POSITION NO.	TITLE	EST. TIME	EST. ANNUAL RATE ($)
1	Project supervisor	¼	4,600
2	Assistant research engineer	¼	3,600
3	Research assistant	½	4,000
4	Electronics system engineer	¼	3,300
5	Technician II (electronics)	½	3,000
6	Stenographer	½	1,700
7	Field supervisor	Full	9,000
8	Technician II	Full	5,500
9	Technician II	Full	5,500
10	Technical assistant	Full	4,000
All			$44,200

6. *Stenographer*

Qualifications: High school graduate, age 20 to 40; must have a minimum of two years of stenographic experience, type a minimum of 70 words per minute, be neat and attractive.

Job description: The stenographer must type all correspondence and reports; keep a record of all purchases, expenditures, personnel time, travel, and salaries; and operate duplicating machines.

MEASUREMENTS TEAM PERSONNEL

7. *Measurements Team Field Supervisor*

Qualifications: College graduate, preferably engineering; preferably single, age 25 to 40, male, have driver's commercial license; a minimum of two years engineering and personnel supervision experience; willing to travel.

Job description: The field supervisor must be an affable, courteous and reliable contact man for the measurements team, be capable of negotiating with those in authority, to represent his employer admirably in the field. He must supervise measurements team field personnel, and supervise operating, testing, and inspection of the electronic, electrical and mechanical equipment relative to nondestructive evaluation of pavements. He is responsible to the project supervisor.

8. *Measurements Team Technician II*

Qualifications: Male, high school graduate, preferably single, age 20 to 40; equivalent of two years experience in electronics, radio and television repair service; must be willing to travel, and have driver's commercial operator license; special military service training in electronics acceptable; must be in good physical condition.

Job description: The technician II will operate, maintain and troubleshoot all mechanical and electronic equipment used by the measurements team (i.e., amplifiers, power supplies, recorders, oscilloscopes); record field data; prepare charts, graphs and/or other engineering forms; drive truck van, station wagon or car which transports the equipment mentioned above to and from test sites. He receives direction from the field supervisor.

9. *Measurements Team Technician II*

Qualifications and job description as above.

10. *Technician Assistant*

Qualifications: Male, high school graduate, age 18 to 40, single; must have driver's commercial operator license; will travel; must be above the average in intelligence and have a high mechanical aptitude; must be in good physical condition.

Job description: The technician assistant will maintain and drive the car or commercial vehicle to and from the test sections and while at the test sections; load and unload any test equipment; maintain, set up and retrieve road blocks for the tests; assist any other member of the measurements team in maintenance and storage of the test equipment. He receives direction and supervision from technician II and from the field supervisor.

CHAPTER SEVEN

RECOMMENDED SAMPLING AND LABORATORY TESTING PROCEDURES

FLEXIBLE PAVEMENTS

Although much effort has been expended by a number of researchers toward the development of laboratory "strength" tests designed to measure the significant properties of flexible pavement materials, no single testing procedure has gained universal acceptance up to this time. Perhaps such a testing procedure will evolve from NCHRP Project 1-10, or from other current research. If such a test is eventually developed, it should be added, of course, to the schedule of tests recommended. Meanwhile, based on the results reported in Chapter Ten and discussed in Chapter One, the ultimate compressive strength of a cylindrical specimen 6 in. in diameter and 8 in. in height and tested at 5-psi lateral pressure in accordance with the standard Texas triaxial test procedure is recommended as the strength test to be used for flexible base, subbase, and subgrade materials. This particular strength test is recommended solely because of the success obtained in the analysis of data from the Texas satellite road test sections. The writers realize that it is entirely probable that a different strength test (for example California bearing ratio or a different triaxial procedure) might have worked as well. However, no other strength data were available for the reported analysis. Details of the recommended procedure, including preliminary determination of the optimum moisture and density, are given in Appendix A.

No firm recommendation for a strength test for the asphaltic surfacing layer can be made at this time. It is, however, recommended that either a compressive strength or a compressional pulse velocity test be selected for this purpose because either of these tests will probably correlate with the triaxial test previously mentioned, and therefore with the "field compression coefficient" discussed in Chapter One. Also, it is recommended that the layer not be remolded and that the strength test be performed for several temperatures so that a strength estimate can be obtained for the in-place material at the time of field measurements. A firm recommendation is not made here because the writers do not know of any existing standard test procedure for measuring either the compressive strength or the compressional pulse velocity of cored pavement layers, and neither time nor funds were available to develop such a procedure.

In more general use than strength tests is a group of relatively simple tests that make it possible to control uniformity of construction, at least to a degree. The reference is to such tests as in situ moisture content and density, gradation, plasticity, etc. Such tests could be used in a national "satellite" program to determine whether the materials used in two or more widely separated test sections that are required (by the experimental study) to be practically identical, do in fact meet that requirement. Conversely, such tests could be used for the purpose of determining whether two test sections, required by the experimental study to contain materials of widely different properties, do in fact differ in that respect. Finally, tests of this type are generally accepted as being related to strength properties, at least qualitatively, and it is possible that they can eventually be correlated quantitatively with performance data gathered in the course of the national satellite program. It is recommended, therefore, that the following tests be performed for each test section:

1. The measurements team should be equipped to perform (with the assistance of local highway department forces) the field determination of in situ moisture content and density of each layer in the pavement structure below the surfacing material. In order to estimate within-section variations these tests should be performed at a minimum

of two locations. Items of equipment required for this work are listed in Table 8; they can be obtained at an approximate cost of $400.

2. In addition, local highway department personnel (under the general supervision of the measurements team field supervisor) should sample at a minimum of two locations and forward material from each layer (other than the surfacing layer) to a central laboratory where the following tests should be performed: (a) gradation, (b) Atterburg limits, (c) triaxial compressive strength at 5-psi lateral pressure, (d) Los Angeles abrasion test (the last named to be performed on base and subbase materials only).

3. With regard to asphaltic surfacing materials, it is recommended that local highway department personnel forward to the central laboratory two undisturbed, 12″ x 12″ (min.) blocks of the surfacing layer, where the following should be determined: (a) strength test (procedure yet to be developed and correlated with triaxial test recommended above for flexible base, subbase, and subgrade materials); (b) specific gravity of the undisturbed sample; (c) bitumen content; (d) grading of aggregates; (e) specific gravity and absorption of fine and coarse aggregates; (f) viscosity of the asphalt. Tests (e) and (f) need to be made on one sample only. If the surfacing material was placed in two or more layers differing in composition, the 12″ x 12″ block should be sawed (in the laboratory) at the interfaces between layers, and the tests recommended above should be repeated for each layer.

It is recommended that all sampling be confined to the wheel paths in the same traffic lane as the test section, but outside its limits in order that the performance of the section will not be affected.

Table 9 summarizes the recommended sampling and testing schedule for one flexible pavement test section. Table 10 lists references to the specifications for each of the recommended tests. It will be noted that Table 9 specifies six sampling points or holes, all to be in the wheel paths near the section, but outside its limits. It is recommended

TABLE 8

FIELD SAMPLING AND TESTING EQUIPMENT REQUIRED BY MEASUREMENTS TEAM

DESCRIPTION	ESTIMATED COST ($)
Field density determination kit:	
Rainhart balloon density apparatus (Soiltest Cat. No. CH-666)[a]	153
Speedy moisture content determination (Soiltest Cat. No. MC-320)[a]	160
Portable oven, drying (Soiltest Cat. No. L-222)[a]	12
Geologist hammer	
Chisels	
Kitchen spoons	
Pans	
Containers for samples	75
Total	400

[a] Source for estimated cost.

TABLE 9

SAMPLING AND TESTING SCHEDULE FOR ONE TEST SECTION (FLEXIBLE PAVEMENT)

SAMPLE HOLE NUMBER [a]	LAYER TO BE SAMPLED	TESTS TO BE MADE ON EACH SAMPLE	WT. OF BOTH SAMPLES (LB)	WHERE TESTED
1–6	—	Measure thickness of layers	—	Field
1, 2	Base	Moisture content, density	30	Field
	Subbase	Moisture content, density	30	Field
	Subgrade	Moisture content, density	30	Field
1, 2	Surfacing	See text	—[b]	Lab.
	Base	Gradation Atterburg limits 5-Psi triaxial strength L.A. abrasion	400	Lab.
	Subbase	Gradation Atterburg limits 5-Psi triaxial strength L.A. abrasion	400	Lab.
	Subgrade	Gradation Atterburg limits 5-Psi triaxial strength	400	Lab.

[a] Odd-numbered holes to be located at one end of section, even-numbered holes at other end.
[b] Each surfacing sample to be 12″ x 12″ x full depth of surfacing, cut with carborundum or diamond saw blade to avoid breaking sample, and packed carefully for shipping to laboratory.

TABLE 10

FLEXIBLE PAVEMENT TEST PROCEDURES

TEST PROCEDURE	NAME OF TEST
ASTM D2167-63T	Density of soil in place by rubber balloon method
AASHO T146-49	Wet preparation of disturbed soil samples for test
AASHO T88-57	Mechanical analysis of soils
AASHO T89-57	Determining the liquid limit of soils
AASHO T90-56	Determining the plastic limit of soils
AASHO T91-54	Calculation of the plasticity index of soils
— [a]	Triaxial compressive strength at 5-psi lateral pressure
AASHO T96-60	Abrasion of coarse aggregate by use of the Los Angeles machine
AASHO T166-60	Specific gravity of compressed bituminous mixtures
AASHO T184-60	Bitumen content of paving mixtures by reflux extractor
AASHO T30-55	Mechanical analysis of extracted aggregates
AASHO T84-60	Specific gravity and absorption of fine aggregates
AASHO T85-60	Specific gravity and absorption of coarse aggregates
ASTM D1856-65	Recovery of asphalt from solution by Abson method
ASTM D2171-63T	Absolute viscosity of asphalts

[a] This test procedure given in Appendix A.

that three holes be dug near each end of the section, and that the thickness of all layers above the subgrade be measured at each hole, in order that within-section variation in layer thickness can be estimated. Four of the holes can be dug with a drill rig; however, one hole at each end will necessarily have to be fairly large in order to perform field testing and also to obtain sufficient material for laboratory testing.

It is recommended that, if possible, the series of tests described above be performed early in the life of the section, but not immediately upon completion of construction. It is felt, based on data from test sections on Texas highways, that significant changes in the strength properties of base materials sometimes occur in the first year of traffic; thus, moisture and density measurements made during or immediately after construction may not properly represent the material as it exists during the major portion of the life of the section. Once an initial series of materials tests has been made on a section, Dynaflect data will furnish a clue as to when additional moisture and density tests might be desirable.

RIGID PAVEMENTS

It is recommended that six 6-in. diameter cores of the portland cement concrete slab be taken in the same traffic lane as the test section, but outside its limits, by local highway department personnel, and that the cores be shipped to the central laboratory for testing. As in the case of flexible pavements, it is recommended that three of the sampling points be located near one end of the section and three near the other end. The cores should be taken with a diamond bit to insure uniformity of diameter.

It is recommended that all sample holes be extended through the subbase material and into the subgrade, and that the thickness of the layers be measured in each hole. The moisture content and density of the subbase and subgrade layers should also be measured in two of the sample holes.

TABLE 11

SAMPLING AND TESTING SCHEDULE FOR ONE TEST SECTION (RIGID PAVEMENT)

SAMPLE HOLE NUMBER [a]	LAYER TO BE SAMPLED	TESTS TO BE MADE ON EACH SAMPLE	WT. OF BOTH SAMPLES (LB)	WHERE TESTED
1–6	—	Measure thickness of layers	—	Field
1, 2	Subbase	Moisture content, density	30	Field
	Subgrade	Moisture content, density	30	Field
1–3	P.C. conc.	Compressive strength Young's modulus Poisson's ratio	—	Lab.
4–6	P.C. conc.	Splitting tensile strength	—	Lab.
Shoulder	Subbase	Gradation Atterburg limits 5-Psi triaxial strength L.A. abrasion	400	Lab.
	Subgrade	Gradation Atterburg limits 5-Psi triaxial strength	400	Lab.

[a] Odd numbered holes to be located at one end of section, even numbered holes at other end.

Representative 200-lb samples of the subbase and subgrade layers should be obtained from the shoulder or dug out from under the pavement edge. These samples obtained from each layer at each end should be shipped to the central laboratory where they are to be tested in the same manner as similar layers in flexible pavements.

In the laboratory all concrete cores are to be measured for length. Young's modulus, Poisson's ratio, and compressive strength are to be determined on three of the cores; the splitting tensile strength is to be measured on the other three.

Table 11 summarizes the recommended sampling and testing schedule for one rigid pavement test section. References to the specifications for each of the recommended tests are given in Table 12.

TABLE 12

RIGID PAVEMENT TEST PROCEDURES

TEST PROCEDURE	NAME OF TEST
ASTM C42-64	Obtaining drilled cores
— [a]	Testing for subbase and subgrade materials
ASTM C174-49	Measuring length of cores
ASTM C469-65	Measuring static Young's modulus and Poisson's ratio of cores
ASTM C42-64	Measuring compressive strength of cores
ASTM C496-64T	Measuring splitting tensile strength of cores

[a] Use appropriate test given in Table 10.

CHAPTER EIGHT

ESTIMATED ANNUAL COSTS

Estimated annual costs for the operation and support of a measurements team are necessarily divided into two phases. Phase A is the initial phase, during which material samples will be obtained from each test section. In this phase, one measurements team, with the assistance of local highway department personnel, can sample materials, perform field

TABLE 13

ESTIMATED ANNUAL BUDGET FOR OPERATION AND SUPPORT OF ONE MEASUREMENTS TEAM DURING OPERATION A (SAMPLING, MATERIALS TESTING, AND NONDESTRUCTIVE TESTING)

Salaries and wages (see Table 7):		
Supervisory and support personnel	$ 20,200	
Measurement team field personnel	24,000	
Total		$ 44,200
Operating expenses:		
Office and field supplies	$ 1,000	
Telephone	1,000	
Equipment maintenance & replacement parts	2,000	
Computer time and programming	5,000	
Travel and subsistence:		
Measurements team (see Table 15)	21,000	
Supervisory personnel	2,000	
Miscellaneous	1,000	
Total		33,000
Miscellaneous costs:		
Equipment (see Table 6)	$ 42,600	
Local highway department personnel:		
Drill rig, flagmen, etc.		
175 sections @ $200/sec.	35,000	
Laboratory testing & material shipments		
175 sections @ $800/sec.	140,000	
Total		217,600
Overhead (40% of salaries and wages)		17,700
Grand total		$312,500

TABLE 14

ESTIMATED ANNUAL BUDGET FOR THE OPERATION AND SUPPORT OF ONE MEASUREMENTS TEAM DURING OPERATION B (NONDESTRUCTIVE TESTING ONLY)

Salaries and wages (See Table 7):		
Supervisory and support personnel	$ 20,200	
Measurements team field personnel	24,000	
Total		$ 44,200
Operating expenses:		
Office and field supplies	$ 1,000	
Telephone	1,000	
Equipment maintenance & replacement parts	2,000	
Computer time and programming	5,000	
Travel and subsistence		
Measurements team (see Table 15)	28,000	
Supervisory personnel	2,000	
Miscellaneous	1,000	
Total		40,000
Miscellaneous costs:		
Local highway department flagmen 525 sections @ $25/sec.	$ 13,000	
Total		13,000
Overhead (40% of salaries and wages)		17,700
Grand total		$114,900

TABLE 15

TRAVEL AND SUBSISTENCE FOR MEASUREMENTS TEAM

Travel and subsistence during Operation A (sampling, materials testing, and nondestructive testing):	
Subsistence $15/day for 4 employees for 7 days	$420
Travel $0.12/mi for 3 vehicles for 500 mi/week	180
Total per week	$600
During this phase one measurements team can process approximately 5 sections and travel about 500 mi per week. Assuming 35 weeks of field operations per year, the annual travel and subsistence expense is $21,000.	
Travel and subsistence During Operation B (nondestructive testing only):	
Subsistence $15/day for 4 employees for 7 days	$420
Travel $0.12/mi for 3 vehicles for 1,050 mi/week	380
Total per week	$800
During this phase one measurements team can process approximately 15 sections and travel about 1,050 mi per week. Assuming 35 weeks of field operations per year, the annual travel and subsistence expense is $28,000.	

tests, and nondestructively test approximately five test sections per week. At this rate, assuming 35 weeks of field operation per year, one team can process about 175 test sections annually. Phase B is the second phase, during which nondestructive testing only will be performed on each test section. One team can nondestructively test approximately 15 test sections per week and therefore process about 525 test sections annually. Estimated annual budgets for Phases A and B are given in Tables 13 and 14 and amount to $312,500, and $114,900, respectively. The budget for Phase A includes the costs for laboratory materials testing and initial costs for the recommended equipment. The average cost per section in Phase A is $1,786; the average cost per section in Phase B is $220.

CHAPTER NINE

FIVE-STATE DEMONSTRATION TRIP

As mentioned in Chapter One, the project research plan included a demonstration of the equipment and techniques of the measurement team in five states to be designated by the sponsor. The states selected were Florida, Alabama, Missouri, Illinois and Minnesota, in each of which a satellite road test program was being conducted by the state. The measurements team traveled to these states, in the order listed, and returned to College Station, Tex., in the period from May 19 to June 9, 1966. By agreement with the sponsor, the team's activities were limited to nondestructive testing. The equipment used, therefore, was the CHLOE Profilometer, the USBPR Roughometer, and the Dynaflect.

The immediate objectives of the trip were as follows:

1. To determine if the team and its equipment could function efficiently over an extended period of time in the field.
2. To determine the within-section variation in the measured variables (deflection and serviceability index) that could be expected in a typical state satellite road test study.
3. To determine whether the measured variables would indicate a significant difference between sections (or designs) in a typical satellite road test study.
4. To compare Dynaflect with Benkelman beam deflections on those projects where the latter were provided by local highway department personnel.
5. To compare the output (slope variance) of the team's CHLOE Profilometer with that of the state's profilometer on projects where the latter was available.

A summary of the findings corresponding to each objective is given in the following, together with references to Tables 16, 17 and 18 summarizing the analyses of variance upon which most of the conclusions are based. The tables are believed to be self-explanatory, except possibly for the symbols used to represent design variables, which are explained as follows:

$D_1, D_2, D_3, \ldots$ = Thickness of surfacing, base, subbase, ...

$S_1, S_2, S_3, \ldots$ = Strength of surfacing, base, subbase, . . .

Tables B-1 through B-7 present details of the analyses of variance summarized in Tables 16 through 18. Those who may be interested in the design details for the projects will find some additional information in Tables B-1, B-2 and B-3 (Florida); Table B-4 (Alabama); Table B-5 (Missouri); Table B-6 (Illinois); and Table B-7 (Minnesota).

SUMMARY OF RESULTS OF DEMONSTRATION TRIP

Efficient Functioning Over Extended Time Period

Team personnel, who had undergone extensive training in the Texas Satellite Road Test, reported no unusual difficulties in making the measurements with the following exceptions:

1. Use of the Roughometer was omitted on some projects because of insufficient section length. (It is estimated that a minimum section length of about 2,000 ft is required to insure reproducible results).
2. Two geophones on the Dynaflect were damaged, but no appreciable delay resulted.

Within-Section Variation, Deflection

Within-section Dynaflect deflection errors, expressed by the coefficient of variation, varied from a minimum of 11% on a Florida project to a maximum of 22% on the Alabama project (Table 16). The average Dynaflect deflection error for all projects was 18%.

Evidence that deflection errors are approximately proportional to the magnitude of the deflection was found in Minnesota, and may be seen by reference to Figure 16, in which the standard deviation of the deflections observed in each Minnesota section is plotted against the mean deflection for the section. The tendency for the error to increase as the deflection increases is clearly demonstrated in this figure. Minnesota's data were chosen for this illustration because of the extremely wide range of deflections and the large number of observations per section.

Within-Section Variation, Serviceability Index

Within-section errors in serviceability index ranged from 2% to 9% for an average of 5%, based on data from three states (Alabama, Missouri, Illinois) (Table 17). In the other two states the test sections had not been divided into subsections, so it was not possible to measure within-section variation.

Between-Sections Variation, Deflection

The difference in Dynaflect deflections between sections (or designs) was highly significant in all states except Florida (Table 16). In Florida, the difference in Benkelman beam deflections between sections just reached significance (Table 16). In Minnesota, the only other state where Benkelman beam data were taken, the difference in Benkelman beam deflections between sections was highly significant.

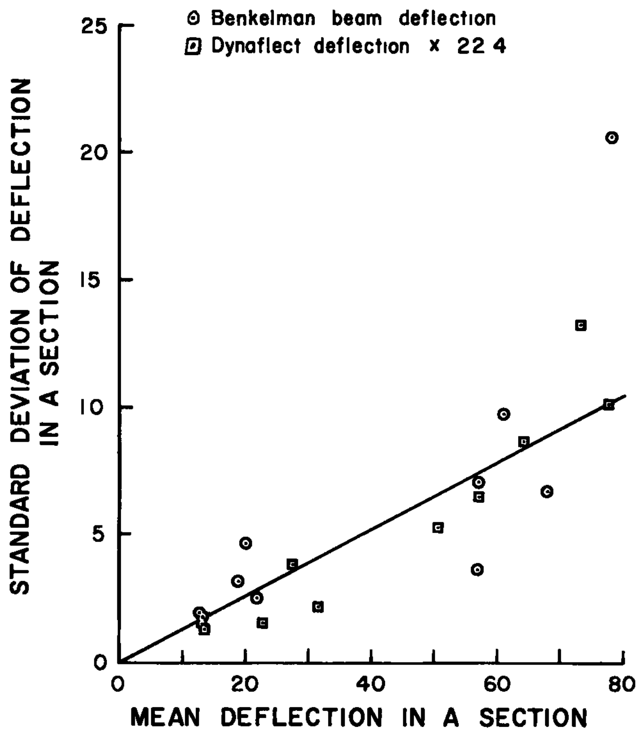


Figure 16. Within-section standard deviation vs mean deflection. Data from Table B-7. Trend line drawn through origin and mean of data.

In summary, it can be said that in nine trials deflections could distinguish between designs in six cases, as may be verified by reference to Table 16. Finding the reasons why deflections did not distinguish between designs in three cases is beyond the scope of this research, because it would require a detailed investigation of the materials involved.

Between-Sections Variation, Serviceability Index

In Alabama the difference in serviceability index between sections was highly significant (Table 17). In Missouri the serviceability index level was very high, averaging 4.6, indicating little change since construction, and, as might be expected, the difference between designs was not significant. In Illinois the general level of serviceability index was somewhat lower, averaging 4.1, but unusually large within-section variation resulted in the difference between sections being not significant. Thus, in two out of three states, the difference in serviceability index between designs was not significant. (As mentioned earlier, sections were not divided into subsections in the other two states, and analyses of variance were not possible).

The failure of the serviceability index to distinguish between designs in two out of three cases reinforces the writers' opinion, based on data from the AASHO Road Test and the Texas satellite road test study, that the serviceability index must be observed over a very long time indeed before significant trends can be discovered, unless the sections are greatly underdesigned.

Comparison of Dynaflect with Benkelman Beam Deflections

Comparisons between Benkelman beam and Dynaflect deflections were made in Florida and Minnesota (Tables B-1 and B-7). Figure 17 plots Dynaflect data (multiplied by a proportionality constant of 22.4 previously established in Texas) against Benkelman beam data. Although consider-

TABLE 16

SUMMARY OF ANALYSES OF VARIANCE, DEFLECTIONS

STATE	PROJ. TYPE [a]	DESIGN VARIABLES	NO. DESIGNS	MEAS. INSTR.[b]	TOTAL OBS.	MEAN DEFL. (0.001 in.)	WITHIN DESIGNS: STD. DEV.	WITHIN DESIGNS: COEFF. OF VARIATION	LEVEL OF SIGNIF. BETWEEN DESIGNS	REFERENCE
Florida:										
Chiefland Proj.	Spec.	D_1, D_2, S_3	18	Dyna.	48	15.8	3.15	20%	NS [c]	Table B-1
	Spec.	D_1, D_2, S_3	12	B.B.	32	17.6	2.32	13%	P<0.1	Table B-1
Marianna Proj., uniform sections	Spec.	D_2, D_3, S_2	6	Dyna.	12	11.8	2.34	20%	NS [c]	Table B-2
Marianna Proj., tapered sections	Spec.	D_2, D_3, S_2	8	Dyna.	16	11.5	1.31	11%	NS [c]	Table B-3
Alabama	Reg.	All	4	Dyna.	60	21.1	4.53	22%	P<0.01	Table B-4
Missouri	Spec.	D_1, D_2, D_3	5	Dyna.	49	26.4	5.67	21%	P<0.01	Table B-5
Illinois	Spec.	D_2, D_3, S_2	3	Dyna.	20	29.7	3.95	13%	P<0.01	Table B-6
Minnesota	Reg.	All	10	Dyna.	110	43.1	6.66	16%	P<0.01	Table B-7
	Reg.	All	10	B.B.	110	40.7	8.18	20%	P<0.01	Table B-7

[a] Spec. = a designed experiment, "special"; Reg. = regular highway sections not constructed as a designed experiment.
[b] Dyna.= Dynaflect; B.B. = Benkelman beam.
[c] Not significant ($P > 0.1$).

TABLE 17

SUMMARY OF ANALYSES OF VARIANCE, SERVICEABILITY INDEX

STATE	PROJ. TYPE [a]	DESIGN VARIABLES	NO. DESIGNS	TOTAL OBS.	MEAN SERV. INDEX	WITHIN DESIGNS STD. DEV.	WITHIN DESIGNS COEFF. OF VARIATION	LEVEL OF SIGNIF. BETWEEN DESIGNS	REFERENCE
Alabama	Reg.	All	4	8	3.5	0.06	2%	$P<0.01$	Table B-8
Missouri	Spec.	D_1, D_2, D_3	5	9	4.6	0.24	5%	NS [b]	Table B-9
Illinois	Spec.	D_2, D_3, S_2	3	9	4.1	0.38	9%	NS [b]	Table B-10

[a] Note a, Table 16. [b] Not significant ($P > 0.1$).

able scatter is evident, good correlation is also evident, and the slope of 22.4 found from Texas data appears to fit the Florida and Minnesota data reasonably well.

Comparison of CHLOE Profilometer with State's Profilometer, Slope Variance

In Florida and Missouri the output of the Texas CHLOE Profilometer was compared with profilometers operated by those states (Table 18). There was no significant difference between the Texas and Florida instruments, but the difference between the Texas and Missouri instruments was highly significant. The slope variance measured by the Missouri instrument was higher than that given by the Texas profilometer by approximately 8%, indicating the desirability of frequent comparison between the various profilometers now in use.

The findings given in the foregoing may be summarized into the following statement of the writers' opinion:

The demonstration trip showed that the team and equipment could operate efficiently. Of the two variables measured—deflections and serviceability index—the former may be expected to furnish valuable information years in advance of the latter. Nevertheless, it is recommended that the measurement of serviceability index in the National Satellite Road Test Program be continued as long as necessary to establish significant trends in pavement performance; otherwise, there will be no means for verifying the potential performance indicated by deflections.

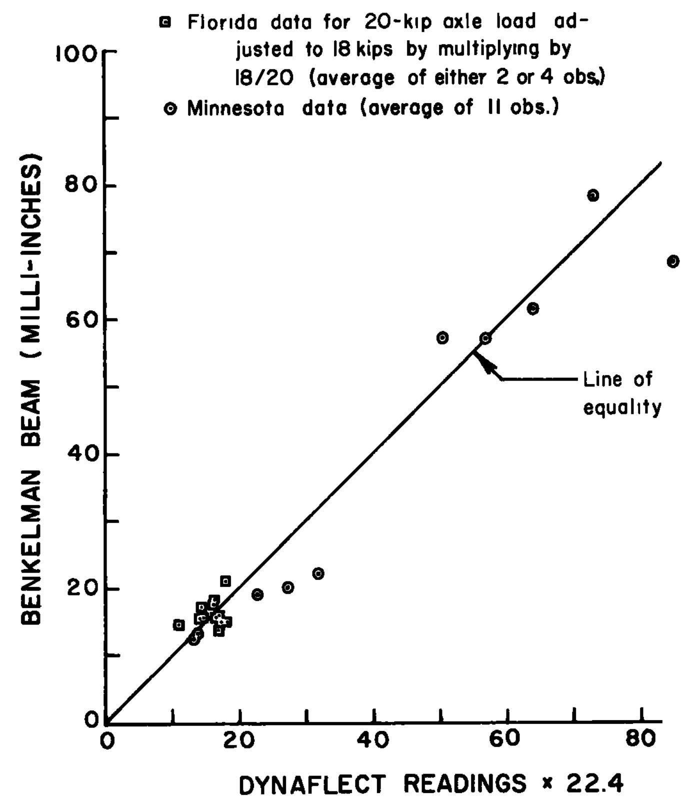


Figure 17. Benkelman beam deflection vs values predicted from Dynaflect. Proportionality constant (22.4) determined on Texas sections.

TABLE 18

SUMMARY OF ANALYSES OF VARIANCE, SLOPE VARIANCE [a]

STATE	PROJ. TYPE [b]	DESIGN VARIABLES	NO. SECTIONS	TOTAL OBS.	MEAN SV [a]	LEVEL OF SIGNIFICANCE BETWEEN SECTIONS	LEVEL OF SIGNIFICANCE BETWEEN WHEEL PATHS	LEVEL OF SIGNIFICANCE BETWEEN CHLOES	REFERENCE
Florida, Chiefland Proj.	Spec.	D_1, D_2, S_3	8	32	4.9	NS	NS	NS	Table B-11
Missouri	Spec.	D_1, D_2, D_3	9	36	4.2	NS	$P<.01$	$P<.01$	Table B-12

[a] Not corrected by the constant normally subtracted prior to use in formula for serviceability index.
[b] See Note a, Table 16.

CHAPTER TEN

EVALUATION OF STIFFNESS OF INDIVIDUAL LAYERS FROM SURFACE DEFLECTIONS

A description of the Lane-Wells Dynaflect and its operation was given in Chapter Four. This chapter describes an assessment of the Dynaflect as a nondestructive means for estimating the relative stiffness of individual layers in a specially designed flexible pavement facility from surface deflection measurements.

The testing necessary to the evaluation was performed on a statistically designed group of experimental pavement sections recently constructed at the Research Annex of Texas A&M University, and financed in part by this research project. Included in this chapter is a description of the facility, the Dynaflect data developed, the development of a mathematical model from elasticity theory, a discussion of the analytical techniques, the results of the analysis, and a discussion of the engineering implications of the analysis.

Appendix C describes the results of laboratory and field measurements of the velocity of compressional waves through the materials used in the pavement test facility.

Appendix D presents a least-squares regression technique that recognizes the existence of errors of measurement in all variables, in contrast to the classical method which assigns all experimental error to the dependent variable. The technique described in Appendix D was used in the analysis work described herein.

PRINCIPAL CONCLUSIONS

The research reported herein led to the following principal conclusions:

1. The analysis of Dynaflect data from the pavement test facility resulted in an equation that predicted the deflection basins (measured in each of 27 sections at five points over a distance of 4 ft) with acceptable accuracy.

2. The eight coefficients appearing in the equation, each related (by assumption) to the stiffness of one of the eight materials involved in the experiment, appeared to be ordered logically, although one coefficient (that for cement-stabilized crushed limestone) had an illogical sign. According to these coefficients, the stiffness of the materials increased in the following order:
   (a) Undisturbed foundation clay.
   (b) Same clay, but compacted (in embankment).
   (c) Sandy clay (in embankment).
   (d) Sandy gravel (in embankment).
   (e) Crushed limestone (in base or subbase).
   (f) Crushed limestone +2% lime (in base or subbase).
   (g) Asphaltic concrete (surfacing material).
   (h) Crushed limestone +4% cement (in base or subbase).

3. Although the eight material coefficients developed in a single analysis of data from the entire facility appeared to be ordered logically, it was not possible to show that logical material coefficients could be found by analysis of data from any single test section. However, work toward that end is continuing in connection with a related research project.

4. Application of the deflection equation to 323 flexible pavement sections on Texas highways indicated that the ratio between observed and predicted values may be a useful index of regional effects on pavement performance.

5. Measurements made on the pavement test facility of seismic wave velocities indicated that "ray theory" seismology was invalid when applied to pavement layers. Nevertheless, velocities determined by the pulse technique are believed to be correct.

6. The multiple-error regression technique used in the analyses has certain advantages over the classical method and should be exploited further. A computer program is available from the writers on request.

PAVEMENT TEST FACILITY

Purpose

The early phases of this research, conducted on existing highways, were dependent on the available design and construction records for interpretation. It was soon clear that if the equipment used by the measurement team was to include nondestructive testing devices, this equipment would have to be evaluated on road sections whose construction characteristics were known as precisely as possible. The field study demonstrated that normal construction records were not precise enough to characterize the instrument responses observed.

Two nondestructive testing systems were already available for evaluation (the Shell Vibrator system and the Dynaflect), with the likelihood that others would be forthcoming. Therefore, it was decided that the construction of a special pavement test facility was warranted.

As a result of these considerations, a test facility was planned and constructed during the spring and summer of 1965. It is known as the Texas Transportation Institute Pavement Test Facility. One-fourth of the cost was paid from NCHRP Project 1-6 funds; the remaining three-fourths was assumed by the Texas Highway Department and the Bureau of Public Roads, co-sponsors of a related research project.

Plan of the Test Facility

The multiplicity of possible cross-sectional configurations made a well-defined plan imperative. The completed facility reflects a selection of configurations based on sound "design of experiment" principles.

Other possibilities with respect to materials, thicknesses of layers, etc., are acknowledged to exist; however, the possible factors were reduced to six chosen so as to be broadly representative of Texas conditions, as well as being applicable to a much wider area. The six factors included in the plan are given in Table 19. Of these, the first three are quantitative in nature and the remaining three are qualitative. The embankment, base and subbase materials are described in Table 20. Considering the materials in light of the ultimate strengths anticipated, it was believed that they could be quantified, at least as to the approximate order of strengths indicated in Table 20. On this basis all six factors to be included in the plan were assumed to be quantitative and it remained to make a selection of thickness and materials combinations.

The cost of building full-scale road sections was estimated to be high. Therefore, the number of combinations had to be kept as small as possible. A general knowledge of the nature of the variables suggested that it would be desirable to be able to investigate at least a second-degree-response surface. These were perhaps the two principal considerations which influenced the choice of design.

The design selected is described as composite (7). Basically, it is made of two parts: (1) a ¼ replicate of a 2^6 factorial—the first 16 treatment combinations (sections) in Table 21, (2) a star consisting of a center point (Section 17) and 12 points on the star (Sections 18 to 29). The six-dimensional space we are working in makes it difficult to envision the nature of the design. However, in a three-dimensional space the corresponding design could be described as the 2^3 factorial portion represented by the corners of a cube, the center of the star being the center of mass of the cube, and the star points lying along lines radiating from the center of mass perpendicular to the faces of the cube. Such a design possesses a number of desirable properties, an important one in this case being the ability to fit a quadratic response surface of the form

$$Y_u = \beta_0 + \sum_{i=1}^{n} \beta_i x_{iu} + \sum_{i=1}^{n} \beta_{ii} x_{iu}^2 + \sum_{i<j}^{n} \beta_{ij} x_{iu} x_{ju} \quad (3)$$

in which n represents the number of factors under study, and obtain estimates of all of the regression coefficients, β.

The general plan of the design is given in the right-hand half of Table 21. Note that any equally spaced set of three levels can be reduced to these ± 1 and 0 coefficients by the following transformation: $(X_i - \bar{x})/\Delta$, where X_i is the actual level, $\bar{x}$ is the mean of the three levels (midpoint), and Δ is the increment between levels. The actual design used in constructing the facility is given in the left-hand half of Table 21, specifying the materials and thicknesses to be used in construction.

Some further economy was effected by constructing only 27 sections, rather than the 29 called for by the design. This was possible because Sections 20 and 22 and Sections 21 and 23 were pairwise physically identical; therefore, only Sections 20 and 21 were actually constructed. On these

TABLE 19

VARIABLES OF TEXAS TRANSPORTATION INSTITUTE PAVEMENT TEST FACILITY

VARIABLE	LEVEL LOW (−1)	MEDIUM (0)	HIGH (+1)
Surface thickness	1 in.	3 in.	5 in.
Base thickness	4 in.	8 in.	12 in.
Subbase thickness	4 in.	8 in.	12 in.
Base material type	4 in.	5 in.	6 in.
Subbase material type	4 in.	5 in.	6 in.
Subgrade material type	1 in.	2 in.	3 in.

TABLE 20

MATERIALS USED IN EMBANKMENT, BASE AND SUBBASE OF TTI PAVEMENT TEST FACILITY

MATERIAL TYPE [a]	MATERIAL DESCRIPTION	WHERE USED	AASHO CLASS.	UNIFIED SOIL CLASS.	TEXAS TRIAXAL CLASS.	COMPRESSIVE STRENGTH [b] (PSI)
1	Plastic clay	Embankment	A-7-6(20)	CH	5.0	22
2	Sandy clay	Embankment	A-2-6(1)	SC	4.0	40
3	Sandy gravel	Embankment	A-1-6	SW	3.6	43
4	Cr. limestone	Base, subbase	A-1-a	GW-GM	1.7	165
5	Cr. limestone +2% lime	Base, subbase	A-1-a	GW-GM	—	430
6	Cr. limestone +4% cement	Base, subbase	A-1-a	GW-GM	—	2270

[a] In assumed order of increasing strength. The foundation material (type 0) and the asphaltic concrete surfacing material (type 7) were not variables in the experiment.
[b] By Texas triaxial procedure, at a lateral pressure of 5 psi.

TABLE 21

EXPERIMENT DESIGN

	ACTUAL DESIGN						THEORETICAL DESIGN [c]					
	LAYER THICKNESS (IN.)			MATL. TYPE [a]			THICKNESS LEVEL			STRENGTH LEVEL		
SEC. NO.	SURF.	BASE	SUBB.	BASE	SUBB.	SUBG.	SURF.	BASE	SUBB.	BASE	SUBB.	SUBG.
1	5	4	4	6	4	1	+1	−1	−1	+1	−1	1
2	1	12	4	6	4	1	−1	+1	−1	+1	−1	1
3	1	4	12	6	4	1	−1	−1	+1	+1	−1	1
4	5	12	12	6	4	1	+1	+1	+1	+1	−1	1
5	5	4	4	4	6	1	+1	−1	−1	−1	+1	1
6	1	12	4	4	6	1	−1	+1	−1	−1	+1	1
7	1	4	12	4	6	1	−1	−1	+1	−1	+1	1
8	5	12	12	4	6	1	+1	+1	+1	−1	+1	1
9	5	4	4	4	4	3	+1	−1	−1	−1	−1	1
10	1	12	4	4	4	3	−1	+1	−1	−1	−1	1
11	1	4	12	4	4	3	−1	−1	+1	−1	−1	1
12	5	12	12	4	4	3	+1	+1	+1	−1	−1	1
13	5	4	4	6	6	3	+1	−1	−1	+1	+1	1
14	1	12	4	6	6	3	−1	+1	−1	+1	+1	1
15	1	4	12	6	6	3	−1	−1	+1	+1	+1	1
16	5	12	12	6	6	3	+1	+1	+1	+1	+1	1
17	3	8	8	5	5	2	0	0	0	0	0	0
18	1	8	8	5	5	2	−1	0	0	0	0	0
19	5	8	8	5	5	2	+1	0	0	0	0	0
20	3	4	8	5	5	2	0	−1	0	0	0	0
21	3	12	8	5	5	2	0	+1	0	0	0	0
22 [b]	3	8	4	5	5	2	0	0	−1	0	0	0
23 [b]	3	8	12	5	5	2	0	0	+1	0	0	0
24	3	8	8	4	5	2	0	0	0	−1	0	0
25	3	8	8	6	5	2	0	0	0	+1	0	0
26	3	8	8	5	4	2	0	0	0	0	−1	0
27	3	8	8	5	6	2	0	0	0	0	+1	0
28	3	8	8	5	5	1	0	0	0	0	0	−1
29	3	8	8	5	5	3	0	0	0	0	0	+1

[a] See Table 20 for description of materials.
[b] Duplicate section, not constructed.
[c] See Table 19 for definition of levels.

two sections it was planned to obtain two independent sets of readings so as to have the set of 29 observations complete for the design. Another decision arrived at in the interests of econômy was to construct the facility as three separate units, with each unit containing 9 sections where a common embankment material was involved. Thus, the final facility was built as in Figure 18, which shows the arrangement of the 27 12-ft x 40-ft test sections, together with five additional sections included in the turn-around at one end of the project, as well as a typical cross section.

TABLE 22

BASE MATERIALS USED IN TURN-AROUND SECTIONS

SECTION NUMBER	BASE MATERIAL TYPE [a]
30	2
31	6
32	4
33	5
34	3

[a] See Table 20 for description of materials.

Special Sections

The turn-around sections constitute a small experiment in which the single variable was base material type, as indicated in Table 22. Neither a subbase nor an embankment was constructed in these sections, and all were surfaced with a two-course surface treatment approximately ¾ in. thick. The base in all sections was 6 in. thick, and the subgrade material was the natural clay, of which only the top 6- to 8-in. layer was compacted.

DYNAFLECT DATA FOR ANALYSIS

Figure 19, a schematic drawing of a block cut out of the pavement test facility, shows the relative positions of the loaded points and the points where deflections were measured by the Dynaflect system. The deflection or geophone

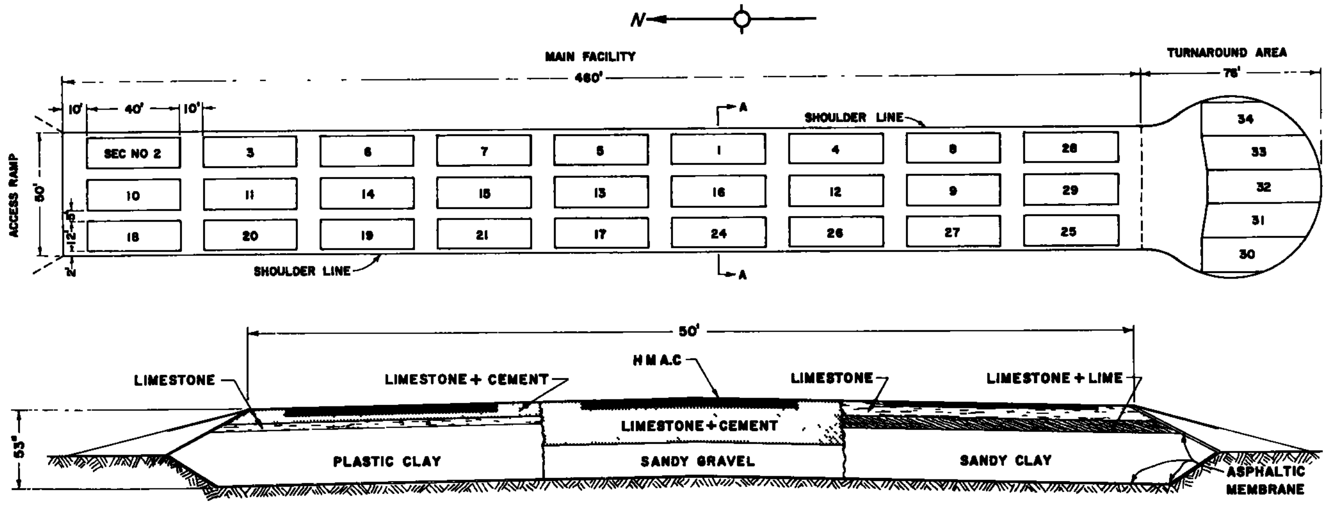


Figure 18. Plan and cross section of TTI Pavement Test Facility.

points, numbered 1 through 5 in the sketch, are identified throughout this report by those numbers.

Although more than usual care was taken in the construction of the test facility to insure as uniform results as possible, tests with the Dynaflect indicated some variation in the deflections measured at different locations on a test section. For this reason, data for the analysis reported herein were taken in each section at six of the eight locations indicated in Figure 20, and were averaged prior to the analysis.

The data were gathered on March 9, 10, 11, and 14, 1966, during the hours from 8 AM to 5 PM. Figure 21 is a plot of air temperatures recorded at Easterwood Airport (located 7 miles from the test facility) during the test period. The average hourly temperature between 8 AM and 5 PM on the four days the testing was carried out was 65.1 F.

Although temperature is known to affect deflection because of its influence on the asphaltic concrete surfacing, the order in which the sections were tested was such as to achieve a randomization of the temperature variable. Thus the effect of temperature should not bias the analysis of the deflection data.

Table 23 gives the averaged Dynaflect data, as well as section design data used in the analysis. The symbols used are the same as those used in the mathematical model described in the next section.

Col. 1 of Table 23, giving the test section identification number, is the same as Col. 1 of Table 19, except that Sections 22 and 23 do not appear (as mentioned earlier, these sections were not constructed).

Col. 2 gives a section index number ($k = 1, 2, \ldots,$ 27).

Col. 3 lists a material index number identifying the materials used in constructing each section ($j = 1, 2, \ldots,$ 7). These numbers (with the exception of 7) are also given in Col. 1 of Table 20, where six of the seven materials used in the test facility are described. The number 7 was assigned to the asphaltic concrete surfacing material, and (as indicated at the bottom of Table 23) the number 0 was assigned to the foundation material below the embankments.

Cols. 4 and 5 show the depths below the surface, H_{1jk} and H_{2jk}, of the top and bottom of the layer composed of material j in section k. (Note that the order in which the layers were constructed in a section is not necessarily the order indicated by the index, j. For example, see Section 5 in Table 23.)

The remaining columns in Table 23 give the deflection, W_{ik}, registered by the ith geophone ($i = 1, 2, \ldots, 5$) on the kth section ($k = 1, 2, \ldots, 27$).

Table 23 presents quantitatively all the information available for analysis, with the exception of a set of dimensions

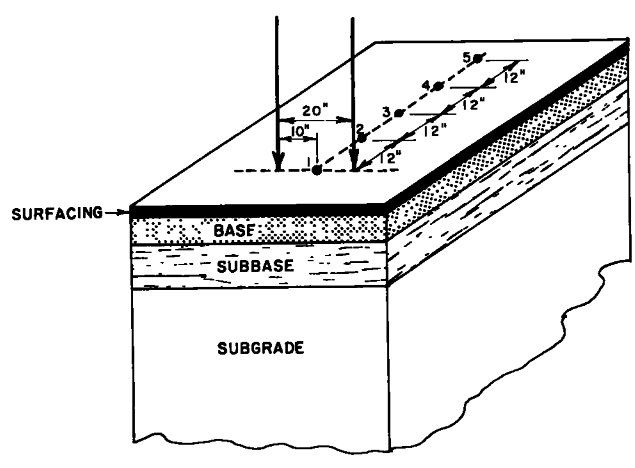


Figure 19. Position of Dynaflect sensors and load wheels. The vertical arrows represent the load wheels; the points numbered 1 through 5 represent the positions where sensors 1 through 5 pick up the motion of the pavement surface.

TABLE 23

DYNAFLECT DATA TAKEN MARCH 9-14, 1966, AND SECTION DESIGN DATA FOR PAVEMENT TEST FACILITY

SEC. NO.	SEC. INDEX, k	MAT'L INDEX,[a] j	DEPTH TO TOP AND BOTTOM OF LAYER[b] (IN.)		DEFLECTION,[c] W_{ik} (MILLI-IN.)				
			H_{1jk}	H_{2jk}	$i=1$	$i=2$	$i=3$	$i=4$	$i=5$
1	1	1	13	53	1.92	1.49	1.01	0.60	0.36
		4	9	13					
		6	5	9					
		7	0	5					
2	2	1	17	53	0.52	0.47	0.41	0.34	0.29
		4	13	17					
		6	1	13					
		7	0	1					
3	3	1	17	53	1.05	0.86	0.58	0.39	0.29
		4	5	17					
		6	1	5					
		7	0	1					
4	4	1	29	53	0.37	0.35	0.32	0.28	0.25
		4	17	29					
		6	5	17					
		7	0	5					
5	5	1	13	53	1.26	0.95	0.61	0.40	0.28
		4	5	9					
		6	9	13					
		7	0	5					
6	6	1	17	53	1.16	0.96	0.72	0.49	0.35
		4	1	13					
		6	13	17					
		7	0	1					
7	7	1	17	53	0.74	0.65	0.53	0.42	0.33
		4	1	5					
		6	5	17					
		7	0	1					
8	8	1	29	53	0.62	0.43	0.33	0.28	0.24
		4	5	17					
		6	17	29					
		7	0	5					
9	9	3	13	53	0.75	0.55	0.38	0.27	0.21
		4	5	13					
		7	0	5					
10	10	3	17	53	0.63	0.50	0.40	0.32	0.26
		4	1	17					
		7	0	1					
11	11	3	17	53	0.63	0.48	0.37	0.28	0.23
		4	1	17					
		7	0	1					
12	12	3	29	53	0.64	0.45	0.32	0.25	0.20
		4	5	29					
		7	0	5					
13	13	3	13	53	0.47	0.43	0.36	0.28	0.22
		6	5	13					
		7	0	5					
14	14	3	17	53	0.40	0.37	0.33	0.27	0.23
		6	1	17					
		7	0	1					
15	15	3	17	53	0.41	0.36	0.32	0.27	0.22
		6	1	17					
		7	0	1					
16	16	3	29	53	0.29	0.26	0.25	0.21	0.19
		6	5	29					
		7	0	5					
17	17	2	19	53	0.72	0.59	0.44	0.33	0.26
		5	3	19					
		7	0	3					
18	18	2	17	53	0.84	0.66	0.48	0.36	0.28
		5	1	17					
		7	0	1					
19	19	2	21	53	0.76	0.64	0.48	0.36	0.28
		5	5	21					
		7	0	5					
20	20	2	15	53	0.73	0.61	0.47	0.35	0.27
		5	3	15					
		7	0	3					
21	21	2	23	53	0.62	0.53	0.42	0.33	0.27
		5	3	23					
		7	0	3					
24	22	2	19	53	0.91	0.67	0.46	0.34	0.26
		4	3	11					
		5	11	19					
		7	0	3					
25	23	2	19	53	0.47	0.43	0.38	0.32	0.26
		5	11	19					
		6	3	11					
		7	0	3					
26	24	2	19	53	0.69	0.58	0.45	0.34	0.26
		4	11	19					
		5	3	11					
		7	0	3					
27	25	2	19	53	0.80	0.67	0.51	0.38	0.30
		5	3	11					
		6	11	19					
		7	0	3					
28	26	1	19	53	1.05	0.86	0.62	0.43	0.30
		5	3	19					
		7	0	3					
29	27	3	19	53	0.51	0.43	0.35	0.28	0.22
		5	3	19					
		7	0	3					

[a] The material index for the foundation on which the embankments were constructed is $j = 0$.
[b] For every test section, $H_{10k} = 53$ and $H_{20k} = \infty$.
[c] Each tabulated deflection is the average of six measurements.

TABLE 24

VALUES OF r_i^2 FOR THE GEOPHONE LOCATIONS

GEOPHONE INDEX, i	r_i^2
1	100
2	244
3	676
4	1396
5	2404

describing the location of each of the five geophones with respect to the loads applied to the pavement by the Dynaflect. Because the arrangement of the geophones and loads was the same on all test sections, and because of the symmetry of this arrangement (Figs. 19 and 20), the dimension selected to describe the location of the ith geophone was the distance, r_i, from that geophone to either of the two loads. Table 24 gives the value of r_i^2 ($i = 1, 2, \ldots, 5$).

The primary purpose of the analysis of the data in Tables 23 and 24 was to provide a basis for accepting or rejecting the following hypotheses:

1. Given the values of W_{ik}, H_{1jk}, H_{2jk} and r_i tabulated in Tables 23 and 24, it is possible to estimate with reasonable accuracy the relative stiffness of the eight materials occurring in the 27 test sections.
2. Given data limited to a single test section, it is possible to estimate with reasonable accuracy the relative stiffness of the four or five materials occurring in that section.

Before performing the analysis it was necessary to construct a trial mathematical model relating the variables named and containing coefficients assumed to be associated with the stiffness property sought from the analysis. The next section is concerned with the construction of such a model.

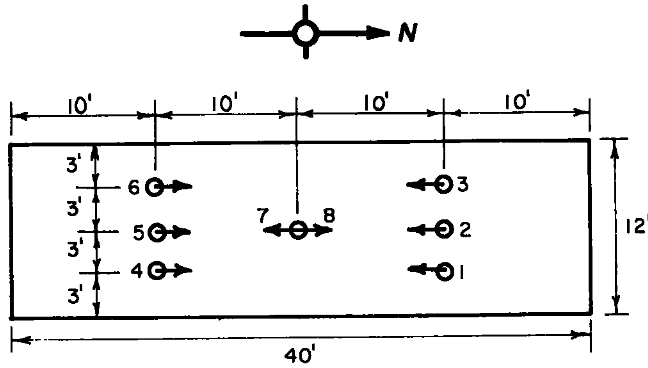


Figure 20. Location of test points in a test section. Dynaflect data were taken with sensor 1 (see Fig. 19) located at one of the test points and the other sensors located in the direction of the arrow.

TRIAL MODEL FOR USE IN ANALYSIS

The deflection at any point in an elastic, layered system can be estimated from elasticity theory, if one is given the thickness, the Young's modulus, and the Poisson's ratio of each layer (8, 9). In the present instance, however, it was required to estimate the Young's modulus of each layer, given the thickness of each and the deflection measured at five points on the surface. The use of rigorous theory for this purpose appeared to be a task of impossible complexity.

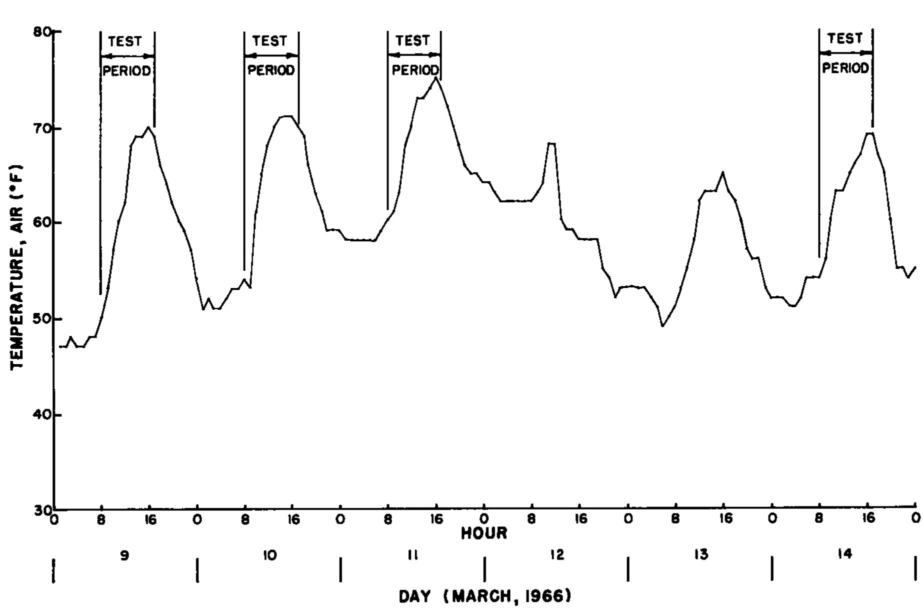


Figure 21. Air temperature during test periods.

It was decided, therefore, to use elasticity theory in part, but to take certain liberties with the theory in order to obtain a usable mathematical model. It was reasoned that if such a model worked—that is, reproduced the measured data with acceptable accuracy and yielded logical values of the coefficients—its use would be justified in spite of the approximations involved in its derivation.

The derivation of a trial model follows:

Consider the deflection produced by a point load. In Figure 22, Q_1 is a point with cylindrical coordinates $r = r_i$ and $z = H_1$ located in the interior of the semi-infinite, homogenous, elastic body bounded by the horizontal plane $z \geq 0$. A second point, Q_2, located directly beneath Q_1, has the coordinates $r = r_i$ and $z = H_2$. A point load, P, acts perpendicular to the boundary plane at the origin of coordinates, 0.

Also, W_1 is defined as the vertical displacement (positive in a downward direction) of point Q_1, W_2 as the vertical displacement of point Q_2, and Δw by

$$\Delta w = W_1 - W_2 \tag{4}$$

in which Δw is the change in length of the line Q_1Q_2 caused by application of the point load, P. Δw is positive if the line Q_1Q_2 is shortened.

Then, according to elasticity theory (*10*), it can be shown that Δw is given by

$$\Delta w = \frac{P}{2\pi}\frac{1+\mu}{E}\left[\frac{2(1-\mu)r_i^2 + (3-2\mu)H_1^2}{(r_i^2 + H_1^2)^{3/2}} - \frac{2(1-\mu)r_i^2 + (3-2\mu)H_2^2}{(r_i^2 + H_2^2)^{3/2}}\right] \tag{5}$$

Consider now a layer of material bounded by the horizontal planes $z = H_1$ and $z = H_2$, as indicated in Figure 22 by the dashed lines, A_1B_1 and A_2B_2. The vertical line Q_1Q_2 is the original thickness of the layer, and Δw is the change in thickness at the horizontal distance, r_i, from the point of application of the load.

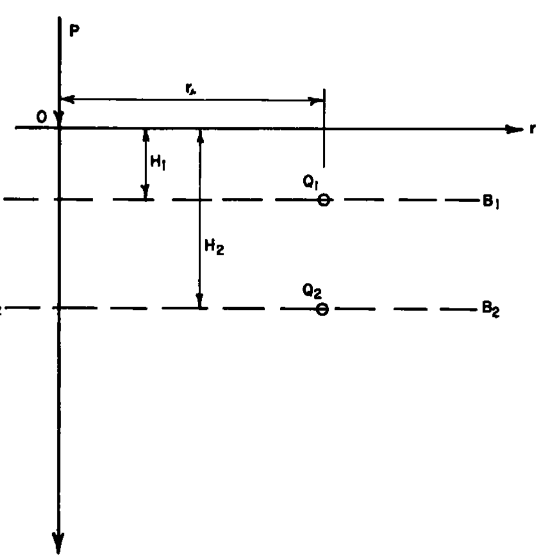


Figure 22. Vertical point load, P, *acting on the boundary plane,* $z \geq 0$, *of a semi-infinite elastic body.*

The half-space in Figure 22 is assumed to be made up of q layers such as the one just described, each layer being of finite thickness except the lowest (foundation) layer, which extends downward to infinity.

Contrary to the previous assumption of homogeneity, and departing from rigorous theory, it is now assumed that the Young's modulus of the material composing any layer, although constant within the layer, differs from that of every other material in the system. A number, represented by j, is assigned to each such material, and j is used as a subscript on symbols representing quantities associated with the jth material ($j = 1, 2, \ldots, q$).

Thus, E_j is Young's modulus for the jth material while H_{1j} is the depth of the upper boundary and H_{2j} the depth of the lower boundary of that material. Δw_{ij} is the change in thickness of the jth material, occurring at the horizontal distance r_i from the point of load application.

Although Young's modulus, E, is assumed to vary from one material to the next, Poisson's ratio, μ, is assumed to be the same for all materials.

Now W_i is defined as the vertical displacement of a point on the pavement surface at the distance r_i from the point of load application. Then W_i is the sum of the Δw_{ij}; that is,

$$W_i = \sum_{j=1}^{q} \Delta w_{ij} \tag{6}$$

in which, according to Eq. 5 and the definitions of subscripts i and j,

$$\Delta w_{ij} = \frac{P}{2\pi}\frac{1+\mu}{E_j}\left[\frac{2(1-\mu)r_i^2 + (3-2\mu)H_{1j}^2}{(r_i^2 + H_{1j}^2)^{3/2}} - \frac{2(1-\mu)r_i^2 + (3-2\mu)H_{1j}^2}{(r_i^2 + H_{2j}^2)^{3/2}}\right] \tag{7}$$

From Figure 19, the distance, r_i, measured from either of the two Dynaflect loads is the same. Thus Eq. 7 applies to either load, and the effect of both loads is obtained simply by multiplying the right side of Eq. 7 by 2.

However, to use Eqs. 6 and 7 as the basis for a model to be used in a regression analysis of Dynaflect data collected on the 27 sections comprising the test facility, it is necessary to introduce a third subscript, k, designating the test section ($k = 1, 2, \ldots, 27$). By attaching the subscript k to the appropriate symbols, the trial regression model is formed as follows:

$$W_{ik} = \sum_{j=0}^{7} A_j X_{ijk} \tag{8}$$

in which

$i = 1, \ldots, 5 =$ geophone index;

$j = 0, \ldots, 7 =$ material index;

$k = 1, \ldots, 27 =$ section index;

$W_{ik} =$ deflection measured by geophone i on section k;

$A_j = \dfrac{P}{\pi}\dfrac{1-\mu^2}{E_j}$

$=$ constants determined by regression analysis (9)

$P =$ load on one load wheel of Dynaflect;

$\mu =$ Poisson's ratio, assumed to be the same for all materials;

E_j = Young's modulus of material j;

$$X_{ijk} = \frac{r_i^2 + B H_{1jk}^2}{(r_i^2 + H_{1jk}^2)^{3/2}} - \frac{r_i^2 + B H_{2jk}^2}{(r_i^2 + H_{2jk}^2)^{3/2}} \quad (10)$$

$$B = \frac{3 - 2\mu}{2(1 - \mu)} \quad (11)$$

r_i = distance from either point of load application to geophone i;

H_{1jk} = depth of upper boundary of material j in section k; and

H_{2jk} = depth of lower boundary of material j in section k.

TECHNIQUE USED FOR DETERMINING COEFFICIENTS A_j

The coefficients, A_j, in Eq. 8 can be determined by classical regression methods for any assigned value for the constant, B. By assigning several values to B, and performing an analysis for each, the particular value of B resulting in the least error in predicting the deflection data can be found. The corresponding values of the coefficients, A_j, then would be regarded as the best estimates of their true values that could be made from the data.

These procedures were used in the investigation of the trial model with one exception: a regression technique different from the classical method was employed, for the reasons advanced in the following.

In the classical method of fitting a linear model to data collected in an experiment involving several variables, it is assumed that the values of all but one—the dependent or response variable—are known precisely. Frequently, however, there are errors of measurement in all the variables. In such cases (the present instance is an example) the classical method yields a biased estimate of the regression coefficients. Inasmuch as the objective of this experiment was to obtain unbiased estimates of the coefficients, A_j, it was apparent that the probability of error in the variables, X_{ijk}, should not be ignored.

The "multiple error regression technique" described in Appendix D makes allowances for measurement errors in all the variables entering into an analysis. Therefore, this technique, rather than the classical method, was used in the analyses reported herein.

The multiple error method requires that the user estimate the ratio of the quality of each controlled variable to the quality of the response variable, the quality of the variable being defined as the ratio of its variance to the variance of the errors made in measuring it. For the analyses reported here, the quality of each variable, X_{ijk}, was taken to be ten times the quality of the response variable, W_{ik}. (In contrast to this 10-to-1 ratio, use of the classical method would require that the ratio be infinite.)

For further details regarding the quality of variables, as well as other features of the multiple error regression technique, the reader is referred to Appendix D, and in particular to the example given at the end of that appendix.

RESULTS OF ANALYSIS

From the data given in Tables 23 and 24, and by use of the procedures outlined previously under "Trial Model for Use in Analysis," the coefficients A_i of Eq. 8 were determined for a wide range of values of the constant, B. Within the logical range of this constant, $1.5 \leq B \leq 2.0$ (corresponding to the full range from zero to one-half of Poisson's ratio), the prediction errors were relatively large. For certain values of B well outside its logical range, the prediction errors were relatively small but the predicted shape of the deflection basins for several of the 27 test sections differed radically from the shapes observed. These results were not considered acceptable. Consequently, the model was altered in several respects so that it would satisfy certain conditions calculated to insure better results. The conditions, three in number, are stated in the following.

With the coefficient, A_j, positive and fixed in value, the compression, $\Delta W_{ijk} = A_j X_{ijk}$, of the layer composed of material j, must satisfy the following inequalities:

$$\frac{\partial \Delta W_{ijk}}{\partial r_i} < 0 \quad \text{(Condition 1)} \quad (12)$$

$$\frac{\partial \Delta W_{ijk}}{\partial H_{1jk}} < 0 \quad \text{(Condition 2)} \quad (13)$$

$$\frac{\partial \Delta W_{ijk}}{\partial H_{2jk}} > 0 \quad \text{(Condition 3)} \quad (14)$$

Condition 1 insures that the surface deflection decreases as r_i increases. Condition 2 insures that the compression of any layer decreases if it is made thinner—or increases if it is made thicker—by lowering or raising its upper boundary. Condition 3 insures that the compression of any layer increases if it is made thicker—or decreases if it is made thinner—by lowering or raising its lower boundary.

The first trial model (Eq. 8) met these conditions only within a limited range of the variables r_i, H_1, and H_2.

The net effect of imposing these three restrictions upon the model was to insure that the deflection basin predicted by the model would always be normal in shape, provided the values of the coefficients, A_j, determined from the data were positive and logically ordered. The second trial model, which conforms to the conditions listed, is

$$W_{ik} = \sum_{j=0}^{7} A_j X_{ijk} \quad (15)$$

in which

$$X_{ijk} = \frac{1}{(cr_i^a + H_{1jk}^a)^b} - \frac{1}{(cr_i^a + H_{2jk}^a)^b} \quad (16)$$

In these expressions, a, b and c are constants, and the coefficients, A_j, are assumed to be inversely related to the resistance of the corresponding materials to compression under load; that is, a relatively small value of A would be associated with a relatively stiff material.

From a series of regression analyses, each performed with a different set of values assigned to a, b and c, the particular set of three constants and eight coefficients that resulted in the least prediction error was found (Table 25).

The prediction error (the root-mean-square residual given in Table 25) was relatively small, being 14.3% of the mean value of the observed deflections.

As may be seen in Table 25, one coefficient, that for cement-stabilized crushed limestone, was illogical in sign. The reason for this is not clear, but it may stem from the

TABLE 25

VALUES OF CONSTANTS IN EQUATIONS 15 AND 16

MATERIAL INDEX, j	COEFF., A_j, IN EQ. 15	MATERIAL	COMP. STRENGTH,[a] S (PSI)	PULSE VELOCITY,[b] V (FPS)
0	1.555×10^{-1}	Plastic clay, undisturbed		
1	1.017×10^{-1}	Plastic clay, compacted	22	2412
2	6.793×10^{-2}	Sandy clay	40	2576
3	3.794×10^{-2}	Sandy gravel	43	3721
4	4.278×10^{-3}	Crushed limestone	165	5222
5	2.804×10^{-3}	Crushed limestone +2% lime	430	5448
6	-1.023×10^{-2}	Crushed limestone +4% cement	2270	7309
7	5.679×10^{-4}	Asphaltic concrete		

Constants in Eq. 16: $a = 1, b = 3/2, c = 3/4$
Root-mean-square-residual in $W = 6.44 \times 10^{-5}$ in.
Mean value of $W = 4.50 \times 10^{-4}$ in.
RMSR (as percent of mean) = 14.3%

[a] Compressive strength at 5-psi lateral pressure, from Table 20.
[b] From Table C-1.

fact that this material was apparently much more rigid than the others, and perhaps contributed so little to the surface deflections that its contribution, represented by the term A_0X_{i6k} in Eq. 15, could not be sensed by the geophones. Under these conditions, normal experimental error in the data could have caused the reversal in sign.

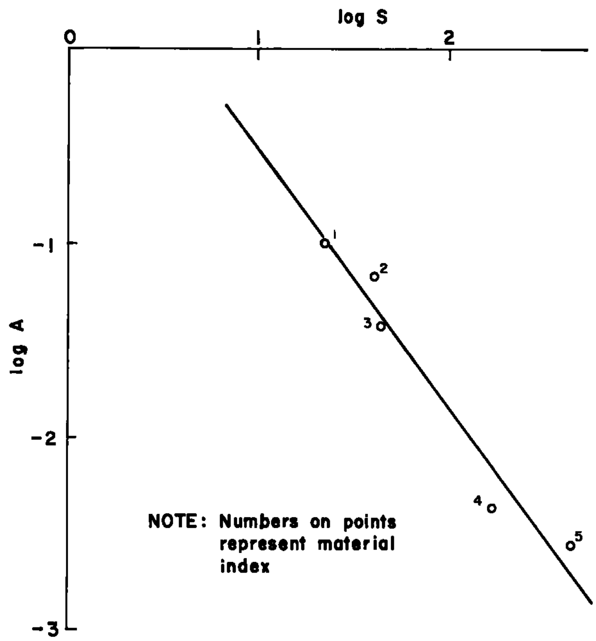


Figure 23. Plot suggesting a functional relationship between the material coefficients, A_j, and the compressive strengths, S_j, given in Table 25.

For comparison with the coefficients, two independent measures of material properties—compressive strength and pulse velocity—are given in Table 25 for the six materials appearing as variables in the experiment design. The compressive strengths were determined from triaxial tests performed at a constant lateral pressure of 5 psi in accordance with standard Texas Highway Department procedures (*4*). The pulse velocities were measured on the compacted materials in place during construction, by the method described in Appendix C, and were necessarily determined before the stabilized materials (types 5 and 6 in Table 25) had completely cured (*11*).

If one interprets the negative coefficient for material 6 as meaning that the coefficient is very small (being algebraically smaller than any of the others), it is apparent from Table 25 that the coefficients A_1 through A_6 occur in the inverse order of compressive strength and pulse velocity, as might be expected. This is taken as evidence that (1) the model does represent the physical phenomena with some degree of accuracy, and (2) the response of the Dynaflect does depend on the stiffness and thickness of the structural components of the pavement.

That a functional relationship exists between the coefficients, A_j ($j = 1$ through 5), and the corresponding compressive strengths, S_j, given in Table 25 is suggested by Figure 23, where log A has been plotted against log S. The straight line was fitted by the multiple error method with the quality of the two variables assumed to be equal. The correlation coefficient was 0.98.

A similar correlation exists between log A and the pulse velocity, V, as indicated in Figure 24. As in the preceding figure, the straight line was fitted by the multiple error method with the quality of the two variables assumed to be the same. The correlation coefficient was 0.99.

The equations for the lines shown in Figures 23 and 24 are, respectively,

$$\log A = 0.8547 - 1.3554 \log S \quad (17)$$

$$\log A = 0.2472 - 5.2596\ V \times 10^{-4} \quad (18)$$

In the derivation of these equations from Table 25, data for material 6 could not be used because the sign of A_6 was negative. Each equation, however, may be used to predict a value of A_6 that will be consistent with the data provided by materials 1 through 5 that were used in deriving the equations. The value predicted from the compressive strength of material 6 is 2.0×10^{-4}, whereas the value predicted from its pulse velocity is 2.5×10^{-4}. Both values are smaller than the coefficient for asphaltic concrete (5.7×10^{-4}), indicating that the latter material was not as stiff as the cement-stabilized crushed limestone. This conclusion is supported by the opinion of members of the project staff who are familiar with the Pavement Test Facility and the materials used in its construction.

Predictions from Eq. 15, using the coefficients and constants given in Table 25, are shown as curved lines in Figure 25 for each section, whereas the observed data are plotted as circled points. The influence of the illogical sign of A_6 is clearly evident in the graphs for Sections 2, 3, 14, 15, and 16.

Also evident from the graph for Section 1 (Fig. 25) is a large discrepancy between the observed data and the predicted deflection basin. The deflections observed on this section are known to be inconsistent with the design of the section, although an investigation into the possible causes of the excessive deflections has not yet been completed. A similar inconsistency, though less pronounced, was discovered in the data from Section 3. Because the data from Sections 1 and 3 had a markedly adverse effect on the trial analyses, deflections from those two sections were not used in the analysis summarized in Table 25. Elimination of these data reduced the total number of observations available for analysis from 135 to 125.

In order to obtain from the coefficients given in Table 25 a prediction equation with logical values for all eight coefficients, a new set of coefficients, A_j', was found by the following two-step procedure:

1. A factor, F, was computed from

$$F = \frac{\overline{W}}{\sum_{j=0}^{7} A_j \overline{X}_j} \quad (19)$$

in which

$\overline{W}$ = the mean of W_{ik} given in Table 25;

A_j = coefficients given in Table 25, except that A_6 is taken as 2.25×10^{-4}, which is the average of the values (2.0×10^{-4} and 2.5×10^{-4}) predicted from compressive strength and pulse velocity by Eqs. 17 and 18;

$\overline{X}_j$ = the mean value of X_j; and

X_j = the value given by Eq. 16 with $a = 1$, $b = 3/2$, and $c = 3/4$.

2. Using the value of F found in step (1), the coefficients, A_j', were computed from

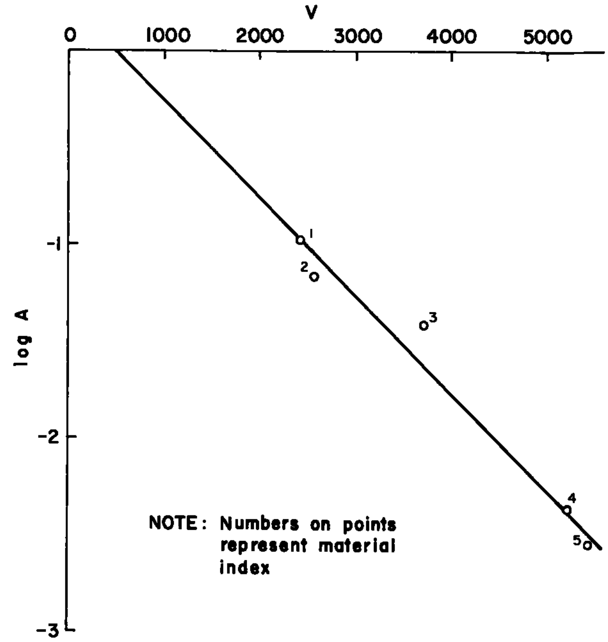


Figure 24. Plot suggesting a functional relationship between the materials coefficients, A_j*, and the pulse velocities,* V_j*, given in Table 25.*

$$A_j' = F A_j \quad (20)$$

This method of transforming the set, A_j, to the set, A_j', preserved the ratios of the coefficients (except ratios involving A_6), and insured that the regression surface defined by the A_j' would—like the surface defined by the A_j—pass through the mean of the data. The transformation was made at some sacrifice in prediction accuracy, but achieved a gain in logic.

The value of F computed from Eq. 19 was 0.9342. Values of A_j' computed from Eq. 20 are given in Table 26. The new prediction equation can be formed by substituting the values of the A_j' in Eq. 15 and giving values of 1, 3/2, and 3/4 to the constants a, b, and c in Eq. 16. Making these substitutions gives

$$W_{ik} = \sum_{j=0}^{7} A_j' X_{ijk} \quad (21)$$

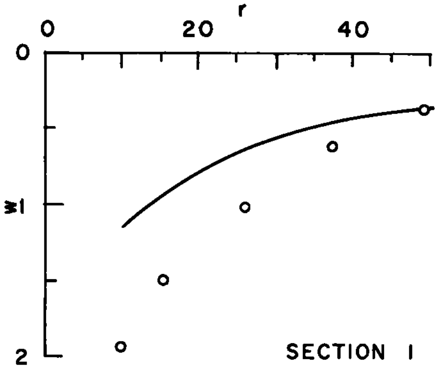
in which

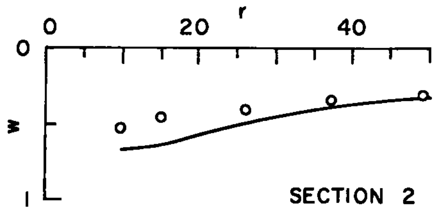
$$X_{ijk} = \frac{1}{(0.75r_i + H_{1jk})^{3/2}} - \frac{1}{(0.75r_i + H_{2jk})^{3/2}} \quad (22)$$

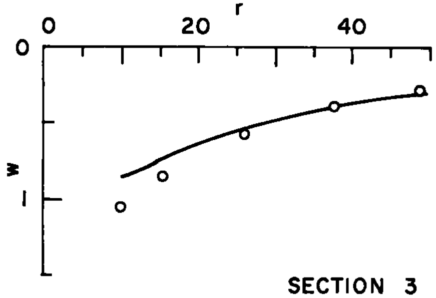
and the A_j' have the values given in Table 26.

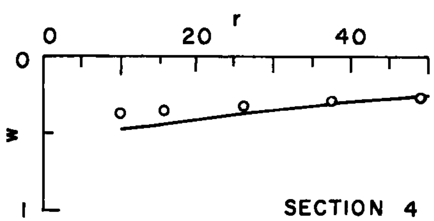
The prediction error for Equation 21 was 19.7% of the mean value of the observed deflections. This may be compared with the error of 14.3% associated with the coefficients, A_j, given in Table 25.

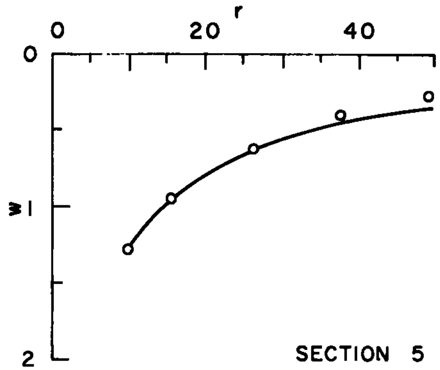
That the transformed coefficients are related to compressive strength and pulse velocity is shown in Figures 26 and 27. The correlation coefficients corresponding to the plotted data were 0.99 in both cases. The equations for the lines shown in the graphs are, respectively,

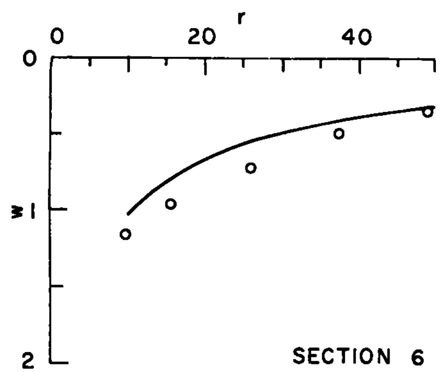






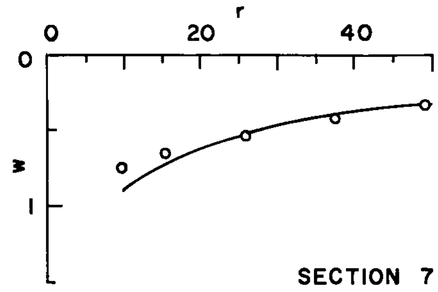


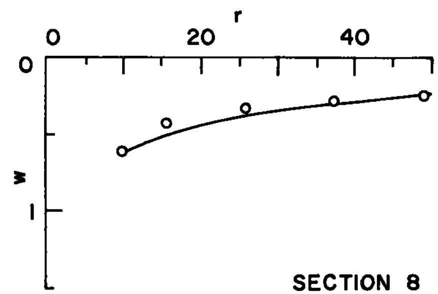


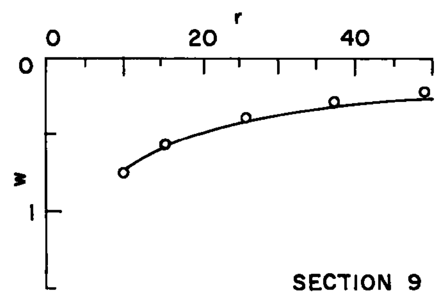


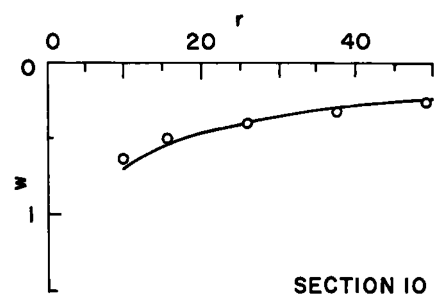
Note: r in inches
w in milli-inches

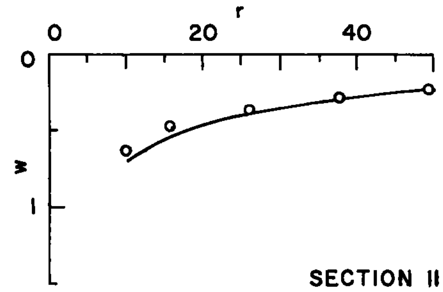
Observed data (circled points) compared with prediction made from deflection equation.

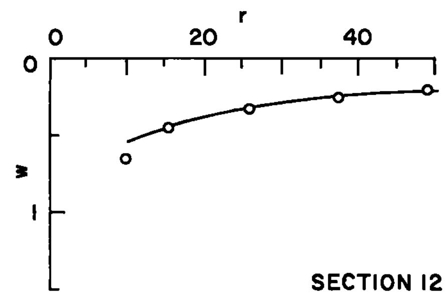


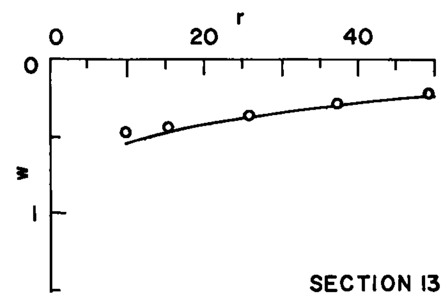












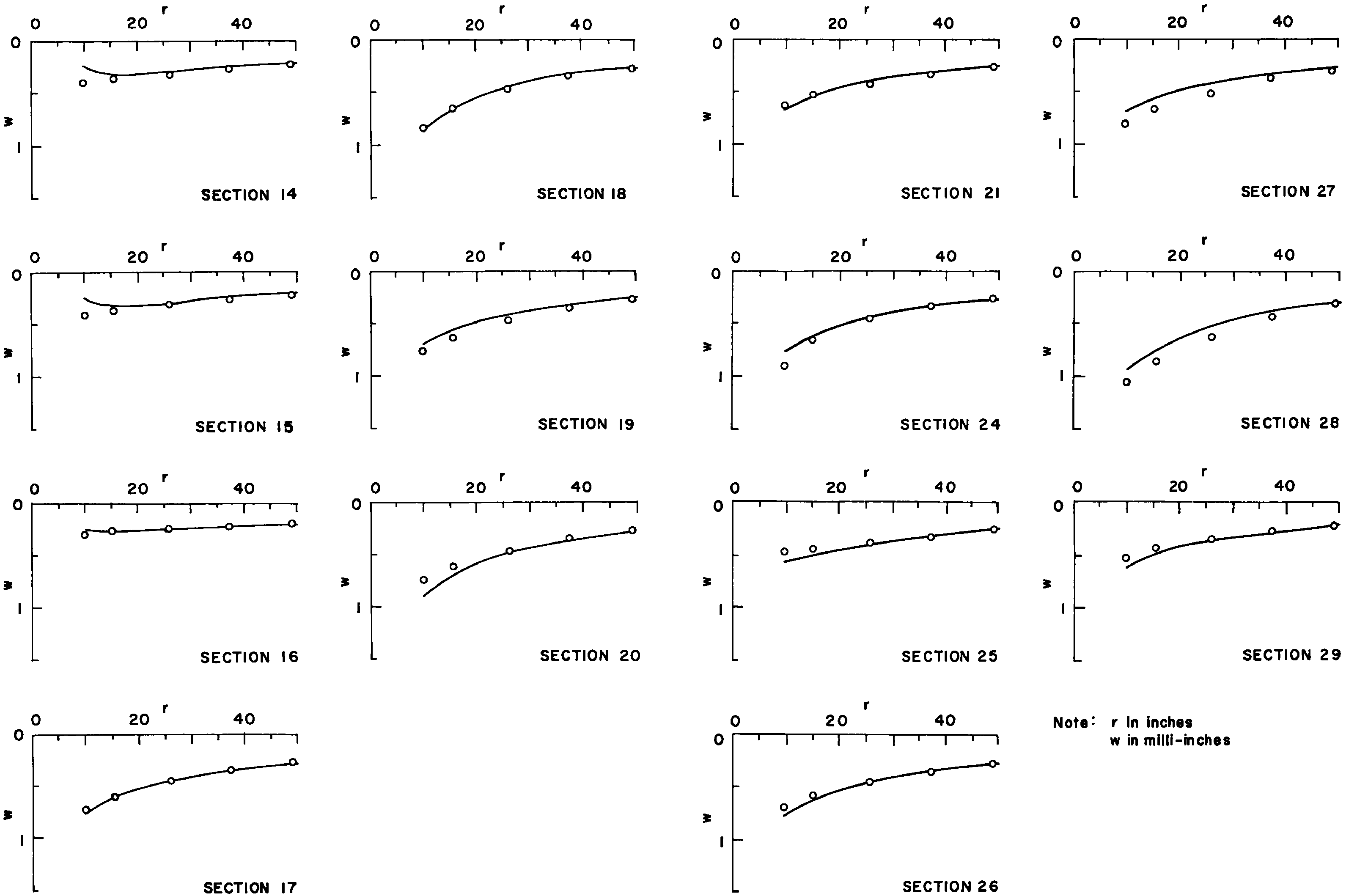


Figure 25. Comparison of A_1 predicted (curves) and observed (points) deflections for the test facility sections.

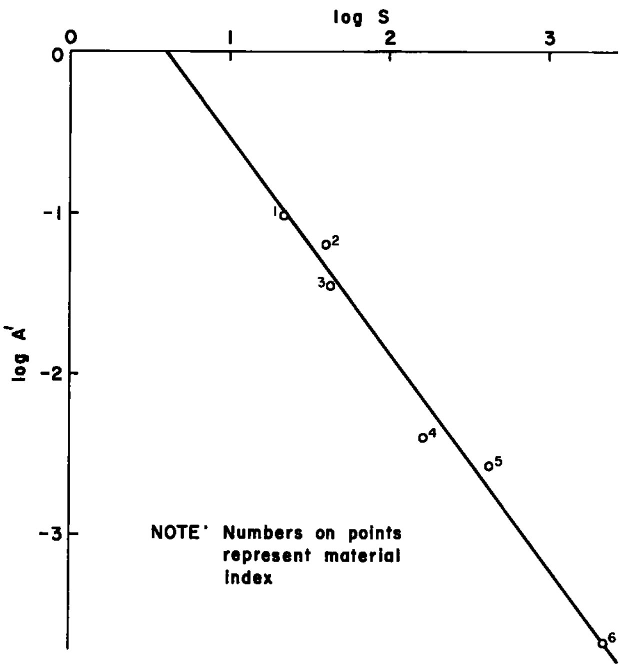


Figure 26. Plot of adjusted coefficients, A_j', versus logarithm of compressive strength.

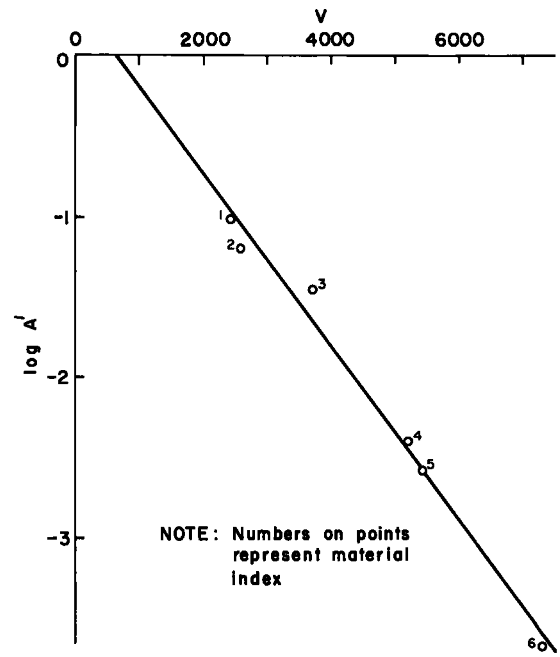


Figure 27. Plot of adjusted coefficients, A_j', versus pulse velocity.

$$\log A' = 0.7911 - 1.3357 \log S \quad (23)$$

$$\log A' = 0.3368 - 5.3766\ V \times 10^{-4} \quad (24)$$

Deflection basins predicted by the transformed coefficients are plotted, together with the observed data, in Figure 28. No anomalies of predicted shape will be found in these figures. For this reason, the transformed coefficients, A_j', are believed to better represent the physical phenomena than the original set of regression coefficients, A_j.

On the basis of the information presented in this section, it is felt that the first of the two hypotheses stated under "Dynaflect Data for Analysis" can be accepted, although future work scheduled in connection with a related research

TABLE 26

VALUES OF A_j' IN EQUATION 21

MATERIAL INDEX, j	COEFF., A_j'	MATERIAL	COMP. STRENGTH,[a] S (PSI)	PULSE VELOCITY,[b] V (FPS)
0	1.453×10^{-1}	Plastic clay, undisturbed		
1	9.505×10^{-2}	Plastic clay, compacted	22	2412
2	6.346×10^{-2}	Sandy clay	40	2576
3	3.544×10^{-2}	Sandy gravel	43	3721
4	3.997×10^{-3}	Crushed limestone	165	5222
5	2.619×10^{-3}	Crushed limestone +2% lime	430	5448
6	2.102×10^{-4}	Crushed limestone +4% cement	2270	7309
7	5.305×10^{-4}	Asphaltic concrete		

Root-mean-square residual in $W = 8.86 \times 10^{-5}$ in.
Mean value of $W = 4.50 \times 10^{-4}$ in.
RMSR (as percent of mean) = 19.7%

[a] Compressive strength at 5-psi lateral pressure, from Table 20.
[b] From Table C-1.

project may provide improved estimates of the relative stiffness of the eight materials.

Attempts to prove the second hypothesis have not, at this writing, been successful, although work directed toward that end is continuing.

ENGINEERING IMPLICATIONS

One objective of the proposed National Satellite Road Test Program is to determine the so-called "regional effect," where a region can be defined as one or more geographical areas within which all pavements of similar design exhibit similar long-time behavior under similar traffic. Herein, however, it is stipulated that the surface deflection of a flexible pavement is an index to its potential long-time behavior under traffic, and the definition of a region is narrowed to that of one or more areas within which all pavements of similar design deflect the same amount under a specified wheel load. That such regions exist within a given area is termed a *regional hypothesis* that can be tested by measuring design parameters and deflections on existing highways in the area.

The regional hypothesis as stated formed the basis for dividing the state of Texas into regions in current research being performed by the Texas Transportation Institute for the Texas Highway Department and the Bureau of Public Roads. With some modifications for northern areas where freeze-thaw cycles cause large seasonal variations in deflections, the hypothesis might also apply to the country at large, and thus might be useful in the National Satellite Road Test Program. This section is therefore devoted to describing the use of the deflection equation (Eq. 21) and other data to delineate regional boundaries in Texas. Also, in this section evidence is presented to prove the regional hypothesis; however, the reasons for the existence of the regions are the subject of current research and are discussed.

For use in the study of regional effects the equations derived as a result of the analysis described in the previous section were altered as indicated in the following.

The starting point is to write equations, based on Eqs. 21, 22, and 23, for the static deflection caused by application of a 9,000-lb wheel load (18-kip single-axle load) to a section in the *r*th region. The equations apply to a four-layer system but may be extended to any number of layers.

$$W_r = 22.4(F_{r1}X_1 + F_{r2}X_2 + F_{r3}X_3 + F_{r4}X_4) \quad (25)$$

in which W_r is the deflection (in units of 0.001 in.) of a section in the *r*th region caused by the application of a 9,000-lb wheel load.

The factor 22.4 is the constant that converts Dynaflect deflections sensed by geophone 1 (see Fig. 19) to estimates of the static deflection caused by a 9,000-lb wheel load.* F_{rj} is the *field compression coefficient* for the material in the *j*th layer of a pavement in the *r*th region (corresponding to the coefficient, A', Eq. 21), and X_j is the *depth coefficient* for the *j*th layer, dependent on the depth below the surface of the top and bottom boundaries of the *j*th layer (corresponding to X_{ijk} in Eq. 22). Expressions for the depth coefficients are taken from Equation 22, and are defined in terms of the layer thicknesses of a section, as follows:

$$X_1 = \frac{1}{(7.5)^{3/2}} - \frac{1}{(7.5 + D_1)^{3/2}} \quad (26a)$$

$$X_2 = \frac{1}{(7.5 + D_1)^{3/2}} - \frac{1}{(7.5 + D_1 + D_2)^{3/2}} \quad (26b)$$

$$X_3 = \frac{1}{(7.5 + D_1 + D_2)^{3/2}} - \frac{1}{(7.5 + D_1 + D_2 + D_3)^{3/2}} \quad (26c)$$

$$X_4 = \frac{1}{(7.5 + D_1 + D_2 + D_3)^{3/2}} \quad (26d)$$

The expression for F_{rj} (see Eq. 27) is a modification of Eq. 23, and is obtained by adding log C_r and log 1,000 to the right side of the latter equation. C_r is a regional factor that is assumed to depend on local environmental and other conditions that can affect the ratio of field strength to laboratory strength. It follows, then, that the environment or conditions at the TTI Pavement Test Facility are the standard for comparison because $C_r = 1.0$ for these conditions. Adding log 1,000 changes the unit of the deflection, W_r, from 1 in. to 0.001 in., the unit most commonly used in pavement research. The resulting equation, in anti-log form, is

$$F_{rj} = \frac{C_r 10^{3.7911}}{S_j^{1.3357}} \quad (27)$$

Eq. 27 serves as the mathematical definition of the regional factor, C_r, because in this equation it is clear that the regional factor modifies the relationship between a field strength, F_{rj}, and a laboratory strength, S_j.

By comparing Eqs. 25 and 27 it can be seen that the former can be written as

$$W_r = C_r W_0 \quad (28)$$

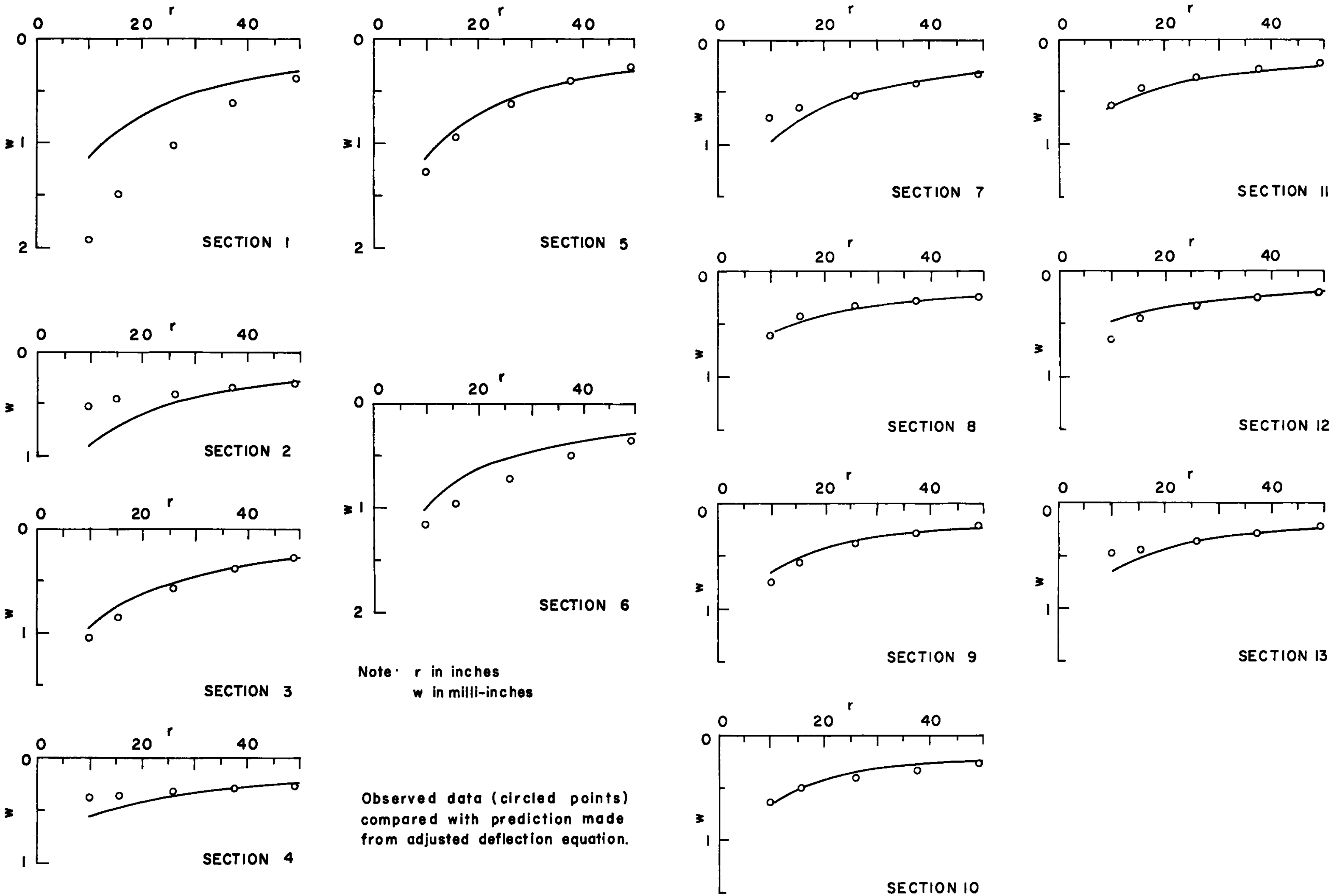
in which W_0 is a deflection computed from Eq. 25 for the special case $C_r = C_0 = 1.0$. Thus, W_0 is an estimate of the deflection that would be observed at the TTI Pavement Test Facility on a section having a given set of design parameters (D_1, D_2, D_3, S_1, S_2, S_3, S_4), whereas W_r is an estimate of the deflection that would be observed on that section if it were located in the *r*th region. It follows that an estimate of the local regional factor is given by

$$\hat{C}_r = W_r / W_0 \quad (29)$$

if W_r is interpreted as an observed deflection in the *r*th region and W_0 as the deflection predicted from Equation 25 for $r = 0$ ($C_0 = 1$).

The data available for testing the regional hypothesis in Texas, the procedures followed, and the rationale of the method, are outlined as follows:

* The derivation of this factor is discussed briefly in Chapter One. See also Figure 17.



Note: r in inches
w in milli-inches

Observed data (circled points) compared with prediction made from adjusted deflection equation.

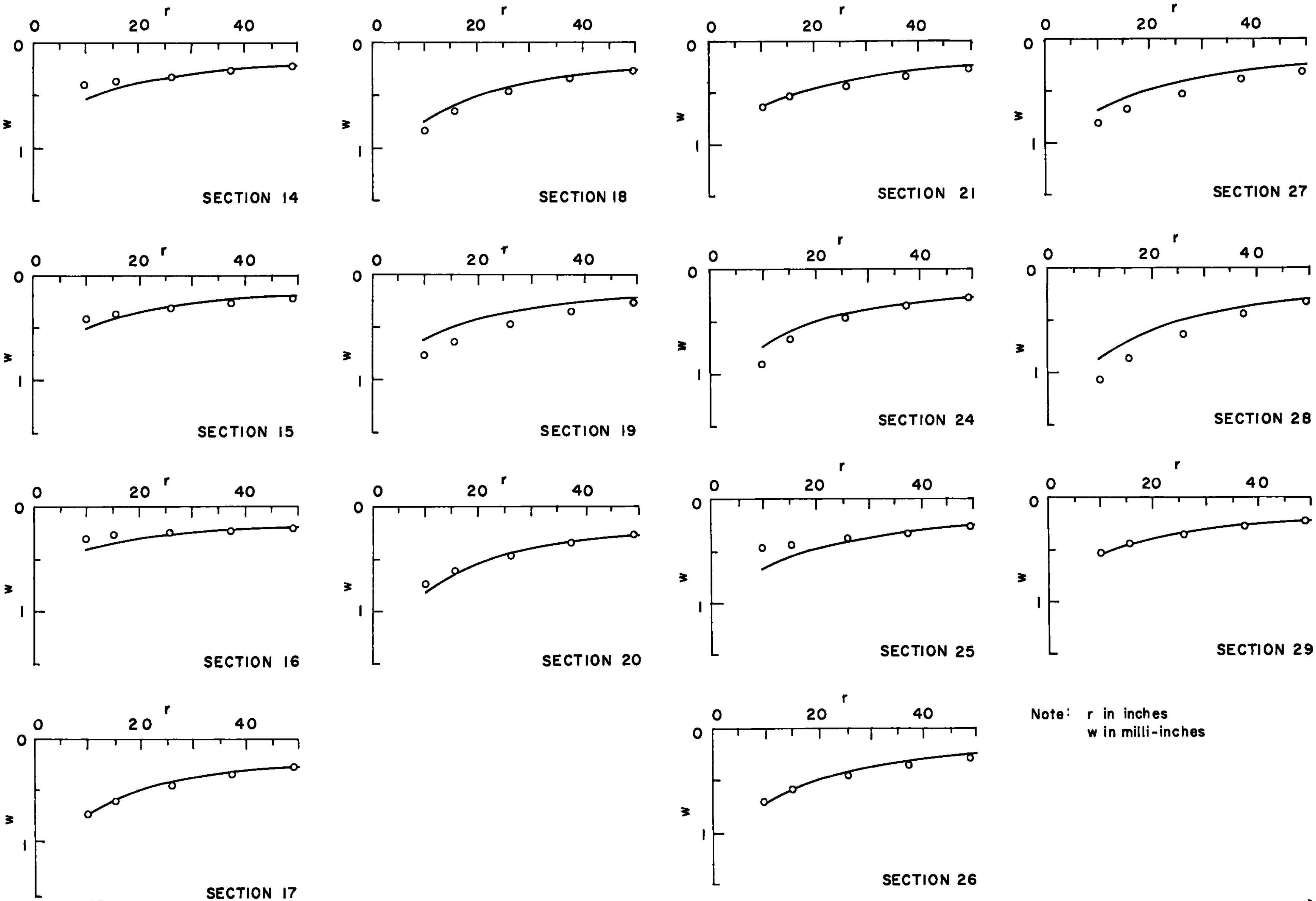


Figure 28. Comparison of A_1' predicted (curves) and observed (points) deflections for the test facility sections.

TABLE 27

DESIGN PARAMETERS FOR ONE TEST SECTION

LAYER NO., i	DESCRIPTION	THICKNESS (IN.)	COMPR. STRENGTH (PSI)
1	Surfacing	D_1	S_1
2	Base	D_2	S_2
3	Subbase	D_3	S_3
4	Subgrade	D_4	S_4

1. The design data of Table 27 (except the compressive strength, S_1, of the surfacing material*) were available for each of 323 flexible sections on Texas Highways.

2. In 1964 deflections were measured by means of the first model of the Dynaflect (single load wheel). These measurements were made at 15 points on each section, averaged, and the average converted to an estimate of the static deflection that would be caused by an 18-kip single-axle load (9,000-lb wheel load), based on the results of field correlation studies relating Benkelman beam to Dynaflection deflections (see Chapter Four).

3. From the data for each test section log $\hat{C}_r$, was computed using Eq. 29. (Log $\hat{C}_r$, rather than C_r itself, was used because of the propensity of the deflection error to be proportional to the deflection, as pointed out in Chapter Nine and illustrated in Figure 16.

4. The values of log $\hat{C}_r$ were written on a map, each value being placed at the location of the test section from which the estimate was computed.

* It not being a routine practice of the Texas Highway Department to measure the compressive strength of asphaltic surfacing materials, the values of S_1 for the 323 test sections were not available. Nevertheless, use of the deflection equation required at least an estimated value of compressive strength for the surfacing layer. The value selected for use for all test sections was obtained by substituting in Eq. 23 the value of A' for asphaltic concrete given in Table 26 (5.305×10^{-4}) and solving for S. The value found by this means was 1,108 psi, which lies between the measured compressive strengths for lime-stabilized crushed limestone (430 psi) and cement-stabilized crushed limestone (2,270 psi) given in Table 26.

5. Contours of equal values of log $\hat{C}_r$ were drawn on the map. The fact that the contours could be drawn was taken as evidence that the quantity, log $\hat{C}_r$, was not randomly distributed geographically, but was in fact related to location.

6. Areas bounded by successive contours of log $\hat{C}_r$ were regarded, tentatively, as regions, and the average value of log $\hat{C}_r$ within each region was taken to be the logarithm of the regional factor, C_r.

Steps 1 through 6 describe what will be termed for discussion purposes as the "main experiment."

Figure 29 shows contours of log $\hat{C}_r$ drawn free-hand (which defined five regions) on a Texas map in accordance with the procedures outlined. Also shown (as small dots) are the locations of the test sections, each of which contributed data from which one value of log C_r was computed. The exact locations of the regional boundaries are not considered definite and where they pass through areas sparsely populated by test sections, the lines are dashed to warn of uncertainty as to their true position.

The contour interval, chosen arbitrarily, was 0.25, and each contour line was taken as the boundary of a region. The result was the division of the state into five regions, two of which were subdivided into several widely separated subregions.

Each region or subregion was assigned a number from 1 to 5, the particular number assigned depending on the value of log $\hat{C}_r$ on the contours forming its boundaries. Table 28 gives the values on the limiting contours of each region, the number of test sections in each region, the average of the values of log $\hat{C}_r$ occurring within the region, and the corresponding anti-log, which was taken as the regional factor.

Table 29 gives the averages for each region of the layer compression coefficients, F_j, the layer depth coefficients, X_j, the measured deflections, W_r, the calculated deflections for the test facility conditions, W_0, and the regional factors. The table shows no significant trends in the design data except that of the subgrade strength, F_4, and this trend is in the wrong direction—the subgrades tend to get weaker as the regions get better.

TABLE 28

SUMMARY OF MAIN EXPERIMENT DATA FROM WHICH LOCATION OF REGIONAL BOUNDARIES AND VALUES OF REGIONAL FACTORS WERE ESTIMATED

REGION, r	LIMITING CONTOURS OF LOG C_r: LESSER VALUE	GREATER VALUE	NO. SECTIONS	AVERAGE VALUE OF LOG C_r	REGIONAL FACTOR, C_r
1	0.25	>0.25	17	0.3732	2.389
2	0	0.25	83	0.1116	1.291
3	−0.25	0	176	−0.1024	0.797
4	−0.50	−0.25	31	−0.3436	0.456
5	<−0.50	−0.50	16	−0.6905	0.204
All			323	−0.0747	0.840

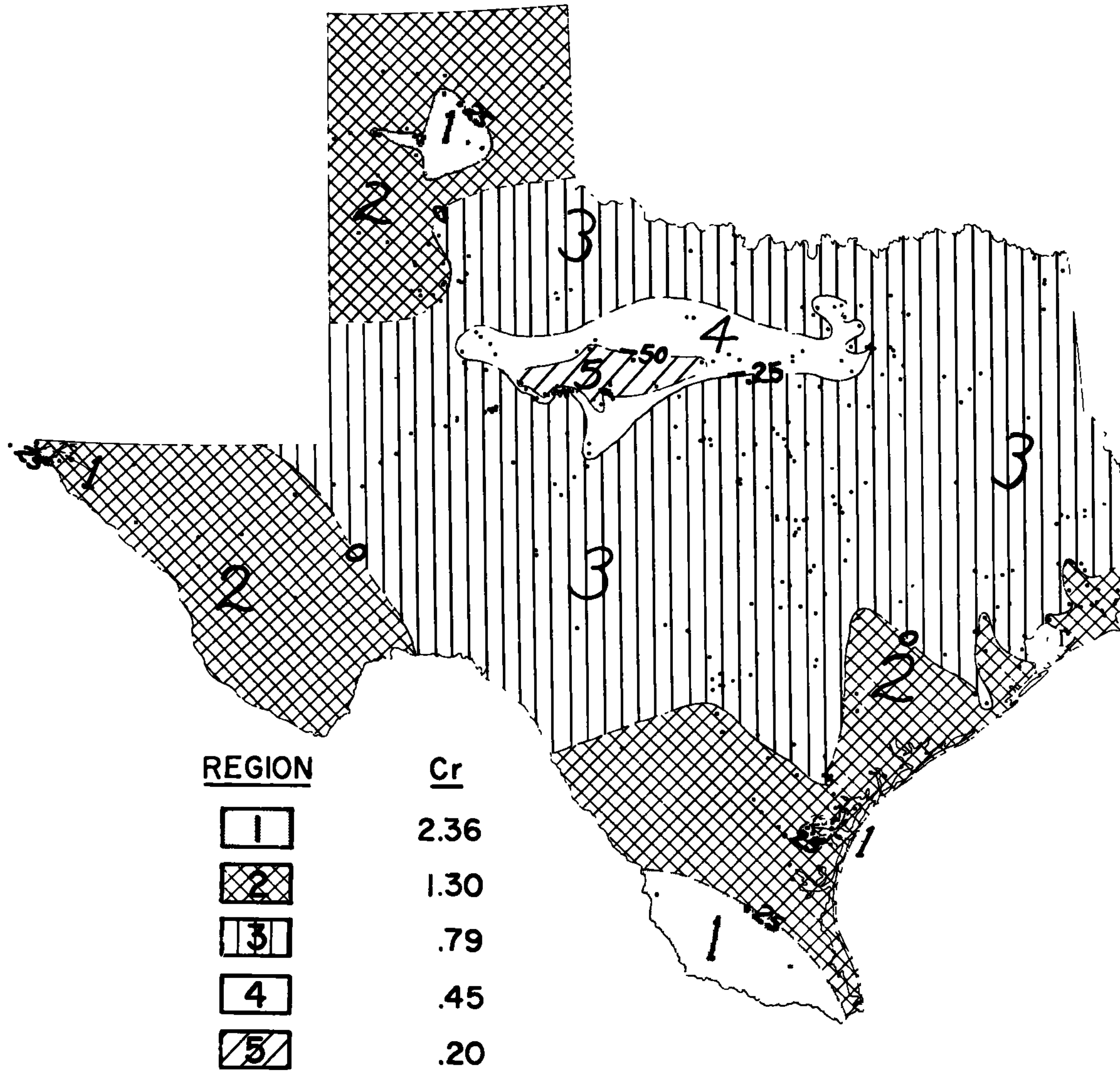


Figure 29. Map of Texas showing regions defining areas of equivalent pavement behavior.

The engineering implications of the wide variation in the regional factor, C_r, become clear if one considers the following.

From Table 28 it can be seen that the deflection observed on a section in Region I can be expected to exceed the deflection that would be observed on a section of similar design at the TTI Pavement Test Facility by a factor in the neighborhood of 2.4 (since $C_1 = 2.389$). On the other

TABLE 29

MAIN EXPERIMENT, REGIONAL AVERAGES OF F_j, X_j, W_r, AND W_0

REGION	COMPRESSION COEFFICIENTS				DEPTH COEFFICIENTS						
	$-F_1$	F_2	F_3	F_4	X_1	X_2	X_3	X_4	W_r	W_0	C_r
1	0.53	13.73	22.63	71.00	0.01381	0.02163	0.00338	0.00986	52.8	22.1	2.389
2	0.53	14.99	26.11	85.47	0.01245	0.02219	0.00409	0.00996	33.3	25.8	1.291
3	0.53	15.01	27.02	104.82	0.01268	0.02218	0.00407	0.00976	23.5	29.5	0.797
4	0.53	16.79	26.40	122.61	0.01298	0.02215	0.00458	0.00897	15.1	33.1	0.456
5	0.53	15.92	23.11	135.63	0.01099	0.02318	0.00386	0.01066	7.4	36.3	0.204

hand, a section in Region 5 would be expected to have a deflection only 20% of that observed on a section of similar design at the test facility ($C_5 = 0.204$). By the same reasoning it can be concluded that (since $C_1/C_5 = 11.7$) a section of a given design in Region I would be expected to deflect more than ten times the amount of a section having the same design parameters but located in Region 5.

In view of the engineering implications just described, it became important to show that variations in the regional

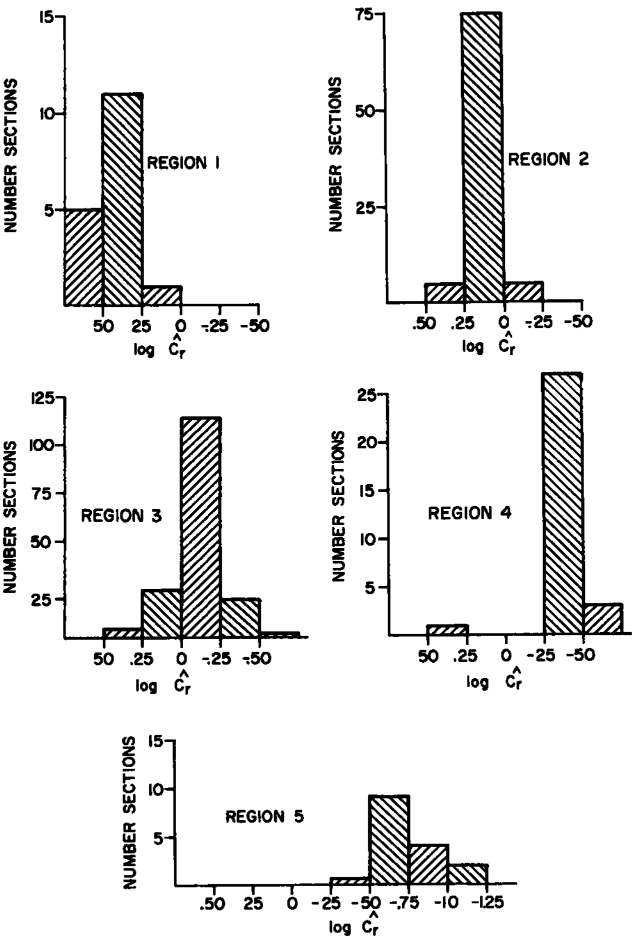


Figure 30. Histogram showing distribution of log $\hat{C}_r$ within regions.

factor, C_r, were neither merely the result of chance nor the result of systematic operational errors. Some evidence that the variations in log $\hat{C}_r$ are related to locality was mentioned earlier under "Trial Model for Use in Analysis." More evidence is presented in Table 30, which gives the results of an analysis of variance. It was concluded from this analysis that the differences in log $\hat{C}_r$ between regions was highly significant when compared to the variation of log $\hat{C}_r$ within regions.. The same conclusion was reached subjectively from an examination of the histograms shown in Figure 30, where the number of sections having a log $\hat{C}_r$ outside the range encompassed by each region is compared graphically with the number inside the range.

The analysis of variance given in Table 31a supplied statistical evidence that the four widely separated subregions of Region 1 did in fact belong to the same region, whereas the analysis shown in Table 31b supported the hypothesis that the three subregions of Region 2 belonged to the same region.

In March 1966 a second set of deflection measurements was made with the 1966 model of the Dynaflect (Fig. 11). This is the same Dynaflect that is discussed in Chapter Four and Appendix F and was used to obtain deflections for the analysis of the A&M Pavement Test Facility described in the preceding sections. The purpose of the second set of measurements was to determine if some type of systematic error contributed to the variations observed in the first set of data. For this investigation measurements were remade on 56 of the test sections scattered over the state. The second set of measurements and the design information associated with these 56 test sections is termed the "1966 Experiment" to distinguish it from "Main Experiment," the larger set of data for 323 sections from which the regional boundaries were established. A summary of the data from the 1966 experiment is given in Tables 32 and 33, which can be compared directly with Tables 28 and 29. This comparison shows a very close agreement between the two sets of data. The results of an analysis of variance for the 1966 experiment are given in Table 34.

It was concluded from the analyses described that the regional hypothesis was valid and that the regions found by the main experiment were in fact highly significant. Similar application of the ratio between measured and predicted deflections may provide a useful index of regional effects in the National Satellite Road Test Program.

TABLE 30

MAIN EXPERIMENT, ANALYSIS OF VARIANCE, LOG $\hat{C}_r$, ALL FIVE REGIONS

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.005)
Total	22.1398	322	0.0687570		
Between regions	14.7333	4	3.68333	158.15	3.85
Within regions	7.40644	318	0.0232907		

TABLE 31

MAIN EXPERIMENT, ANALYSIS OF VARIANCE, LOG $\hat{C}_r$

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.05)$
(*a*) SUBREGIONS OF REGION 1					
Total	0.138167	16			
Between subregions	0.019237	3	0.006412	0.70	3.41
Within subregions	0.118930	13	0.009148		
(*b*) SUBREGIONS OF REGION 2					
Total	0.89365	82			
Between subregions	0.015712	2	0.007856	0.72	3.11
Within subregions	0.877939	80	0.010974		

TABLE 32

SUMMARY OF 1966 EXPERIMENT DATA

(COMPARE WITH TABLE 28)

REGION, r	LIMITING CONTOURS OF LOG C_r: LESSER VALUE	GREATER VALUE	NO. SECTIONS	AVERAGE VALUE OF LOG C_r	REGIONAL FACTOR, C_r
1	0.25	>0.25	3	0.3092	2.038
2	0	0.25	10	0.0494	1.121
3	−0.25	0	34	−0.0697	0.852
4	−0.50	−0.25	8	−0.3151	0.484
5	<−0.50	−0.50	2	−0.7016	0.199
All			56	−0.0870	0.818

TABLE 33

1966 EXPERIMENT, REGIONAL AVERAGES OF F_j, X_j, W_r, AND W_0

(COMPARE WITH TABLE 29)

REGION	NO. SECTIONS	COMPRESSION COEFFICIENTS F_1	F_2	F_3	F_4	DEPTH COEFFICIENTS X_1	X_2	X_3	X_4	W_r	W_0	C_r
1	3	0.53	14.19	28.79	74.44	0.01776	0.01800	0.00432	0.00860	44.867	20.743	2.038
2	10	0.53	12.82	34.45	106.77	0.01487	0.02066	0.00437	0.00878	32.970	29.240	1.121
3	34	0.53	14.34	28.39	100.67	0.01527	0.02186	0.00343	0.00813	23.153	26.583	0.852
4	8	0.53	15.15	23.64	118.20	0.01291	0.01950	0.00794	0.00833	16.912	34.472	0.484
5	2	0.53	16.01	20.22	196.70	0.01484	0.01975	0.00594	0.00815	8.300	46.588	0.199

TABLE 34

1966 EXPERIMENT, ANALYSIS OF VARIANCE, LOG C_r, ALL REGIONS

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.005)$
Total	4.0634	56	0.072561		
Between regions	1.83866	4	0.459664	10.74	4.24
Within regions	2.22477	52	0.042784		

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APPENDIX A

TRIAXIAL COMPRESSIVE STRENGTH AT 5-PSI LATERAL PRESSURE

The purpose of this appendix is to describe the procedure that should be followed for the determination of the triaxial compressive strength at 5-psi lateral pressure of samples of base, subbase and subgrade materials. The procedure involves two areas; the determination of the optimum moisture content and the testing of a sample in compression. The procedure for the optimum moisture content determination is taken from test method TEX-113-E and the procedure for triaxial compressive strength determination is taken from test method TEX-117-E. Both of these procedures can be found in the "Manual of Testing Procedures" of the Texas Highway Department (*4*).

LIST OF APPARATUS

1. Sieves: Standard U. S. woven wire sieves with square openings (ASTM E 11 specifications) 1¾-in., 1¼-in., ⅞-in., ⅝-in., ½-in., ⅜-in., No. 4, and No. 10.
2. Heavy-duty scale: A scale with 500-lb capacity and sensitive to 0.5 lb.
3. Scale: An aggregate scale with 35-lb capacity and sensitive to 0.01 lb.
4. Electric air dryer with temperature range of 130-230 F.
5. Scoop.
6. Sample containers, metal pans, cardboard cartons.
7. * Automatic tamper: A compaction device with base plate to hold 6-in. I.D. forming molds, equipped with 10-lb ram and adjustable height of fall. Striking face of 10-lb ram, 40° segment of 3-in. radius circle. No. 662.
8. * Compaction mold, 6-in. I.D. and 8½-in. height, with removable collar. No. 665565.
9. * Measuring device, micrometer dial assembly for determining height of specimens, with set of standard spacer blocks. No. 725 MA.
10. Extra base plate to hold forming mold.
11. Press, to eject specimens from mold.
12. * Circular porous stones slightly less than 6 in. in diameter and 2 in. high. No. 720 PS.
13. A supply of small tools: hammer, plastic mallet, level, etc.
14. * Axial cells, lightweight stainless steel cylinders; 6¾-in. inside diameter and 12 in. in height, fitted with standard air valve and tubular rubber membrane 6 in. in diameter. No. 720.
15. Aspirator or other vacuum pump.
16. Air compressor.
17. Damp room equipped with shelves and supply of constant-pressure air.

* Special equipment items obtainable from Rainhart Co., Austin, Tex.

18. *† Screw jack press and assembly. No. 725E.
19. *† Dial housing and loading block to transmit load to cylindrical specimen (part of item 18).
20. *† Calibrated proving rings according to ASTM Designation E4-57T. No. 725R15.
21. Micrometer dial gauge, calibrated in 0.001 in., with support to measure deflection of specimen.
22. Pressure regulator, gauges, and valves.
23. Circumference measuring device, special-made metal tape measure.
24. Lead weights for surcharge loads.
25. Rectangular stainless steel pans 9 in. by 16 in. by 2¼ in. deep equipped with porous plates.

DETERMINATION OF OPTIMUM MOISTURE-DENSITY

Preparation of Material

1. Calibration of Equipment.—Specimens approximately 6 in. in diameter and 8 in. in height are molded in performing this test. The compacted material is not trimmed with a straight edge and does not completely fill the mold. Therefore, it is necessary to determine the volume per linear inch of height for the mold. (a) Measure the diameter of the mold, by means of the micrometer caliper and micrometer dial, at the ends and several intermediate points to obtain an average value for the diameter. (b) Use the average diameter to calculate a mean cross-sectional area of the mold. (c) Calculate the volume, in cubic feet, for 1 in. of height of the mold.
2. A representative sample of at least 100 lb should be spread on a clean surface and air dried. Subgrade material that will dry in lumps should be crushed with a tamping device.
3. Base course and other aggregates should then be separated into appropriate sizes by dry screening. The entire sample should pass the 1¾-in. sieve. Other sieve sizes that give the desired gradation are 1½-in., ⅞-in., ⅝-in., ⅜-in., No. 4, and No. 10. Each size material should be weighed to arrive at the percentage of material retained on each sieve for use in recombining the material into an individual sample.

Recombining Material and Compaction Procedure

1. Estimate the weight of dry material that will be needed to obtain an 8-in. sample when wetted and compacted in the 8½-in. high mold. By using the estimated weight and the percent retained of each size material as

† Not necessary if a suitable testing machine is available. The testing procedure remains essentially the same except that the methods for obtaining vertical load and deformation have to be modified.

determined earlier, the cumulative weights of each size material needed to recombine a sample can be computed.

2. Weigh out a sample, using weights obtained in step 1. Estimate the percent of moisture at optimum and calculate the water to add to the sample based on the weight of dry soil. Water and material are then mixed by hand or with a mechanical mixer.

3. Place one-fourth of the sample in the mold and compact it and each successive layer the required number of blows. For granular materials use 50 blows per layer. For other materials (except cohesionless sands) use the following schedule:

P.I.	Blows/Layer
20–35	25
35–45	15–20
>45	15

For a cohesionless sand a twin-head 10-lb ram is used, which must be padded with sponge rubber. One hundred blows are used for each of eight 1-in. layers. Repeat these steps until the entire 8-in. sample has been compacted.

4. Finish the top of the sample until a level surface is obtained. Obtain the height of the sample, extrude the sample, and obtain the wet weight. Put the sample in a tared pan, break up the material and dry.

5. Repeat steps 1 through 4, varying the amount of water added to obtain points for a moisture-density curve. Calculate the moisture and the density and plot a moisture-density curve.

TRIAXIAL COMPRESSIVE TESTING

Preparation for Capillary Absorption

1. Determine the optimum moisture content from the moisture-density curve. Mold one sample at the optimum moisture-density.

2. After extruding the sample, place a 6-in. diameter by 2-in. high porous stone on the top and the bottom of the sample. Dry the granular material sample in an oven at 140F for 3 to 6 hr. For very plastic clay, air dry at room temperature until surface cracks have formed to ¼-in. depth. For moderately active soils, oven dry at 140F; if cracks appear, allow the sample to air dry at room temperature. Allow the sample to cool to room temperature and weigh the sample plus the porous stones. After the sample has been dryed and weighed, wrap it in filter paper and place it in an axial cell for capillary absorption.

Capillary Absorption

1. Place the sample in a pan and add water to within ½ in. of the bottom of the sample. Connect the axial cell to the air manifold, open the valve to apply a constant 1-psi lateral pressure, and maintain this throughout the period of absorption.

2. For granular material a vertical surcharge load of 0.5 lb per square inch of end area of the sample should be placed on the top porous stone. Consider the top stone as part of the surcharge load. For subgrade soils the surcharge should be 1 lb per square inch.

3. For base materials and soils with P.I. of 15 or lower, allow 10 days of absorption. For material with P.I. above 15, use the number of days equal to the P.I. of the material for capillary absorption.

4. After absorption is completed, remove the axial cell and the filter paper. Record the circumference of sample. Replace the axial cell and the sample is ready for testing.

Testing Specimens

In brief, the specimens are tested in compression while being subjected to a constant lateral pressure of 5 psi. The motorized press should be geared to travel at a rate of 0.13 to 0.15 in. per minute. Simultaneous readings of load and deformation are taken at intervals of 0.01-in. deformation until the specimen fails.

1. Disengage the worm gear drive and crank the press down far enough to have room to place the specimen, the metal loading blocks, and the special bell dial housing in the press.

2. Center the specimen with the upper and lower metal loading blocks in place in the press. Adjust the deformation gauge in such a manner that it will be down against the center of the top spacer block and also compressed for almost the length of travel of the stem. The gauge must be placed in this position because the specimen moves away from the gauge during the compression. Set the dial of the strain gauge to read zero.

3. Next, set the bell housing over the strain gauge and adjust it so that it does not touch the gauge or its mounting. At this point it should be noted that the compressive stress will necessarily be applied along a vertical line through the center of the ball that is mounted in the top of the bell housing. Because it is desirable to apply the compressive force along the vertical axis of the test specimen, shift the bell housing laterally to bring the ball directly over the axis of the specimen. Raise the press by means of the motor, align and seat the ball on the bell housing into the socket in the proving ring. Then apply just enough pressure to obtain a perceptible reading on the proving ring gauge. Read the strain gauge and record as deformation under dead load.

4. Connect the air line to the axial cell and apply a lateral pressure of 5 psi to the specimen. The lateral pressure applied by the air will tend to change the initial reading of the strain gauge. As the air pressure is adjusted, start the motor momentarily to compress the specimen until the deformation gauge reads the same as recorded in step 3. Read the proving ring gauge and enter in the load column opposite the initial deformation reading.

5. The test is ready to be started. Turn on the motor and read the proving ring dial at each 0.01-in. deformation of the specimen. Continue reading until 60 readings have been taken unless failure occurs earlier. Failure is reached when the proving ring dial readings remain constant or

decrease with further increments of deformation. In testing specimens with aggregates, the slipping and shearing of aggregates will cause temporary decreases in proving ring readings. The test should be continued until true failure is reached. After 60 readings the cross-sectional area of the specimen has increased so that the subsequent small increase in load readings is little more than the increase in tension of the membrane acting as lateral pressure.

6. Calculation of strength at 5-psi lateral pressure. Using the circumference of the sample obtained after capillary absorption, calculate the cross-sectional area of the sample. Then divide the load at failure by the area of the sample. The resulting figure is the compressive strength at 5-psi lateral pressure.

APPENDIX B

ANALYSES OF VARIANCE OF DATA TAKEN ON FIVE-STATE DEMONSTRATION TRIP

TABLE B-1

FLORIDA, CHIEFLAND PROJECT

(*a*) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES [a] D_1	D_2	S_3 [b]	SECTION NO. REP. 1	REP. 2	DEFLECTION (0.001 IN.) DYNAFLECT NO. OBS.	MEAN	BENKELMAN BEAM [d] NO. OBS.	MEAN
1	2	6	L	3N	10N	4	17.9	2	18
2	2	6	M	5S		2	11.0	2	16
3	2	6	H	8N		2	16.3	2	20
4	2	9	L	1N [c]		2	13.6	0	—
5	2	9	M	6N	11N	4	18.2	4	16.5
6	2	9	H	9S		2	14.7	2	17
7	2	12	L	2S [c]		2	15.5	0	—
8	2	12	M	4N [c]		2	16.4	0	—
9	2	12	H	7S	12S	4	17.0	4	15
10	3	6	L	3S	10S	4	14.9	2	19
11	3	6	M	5N		2	14.2	2	17
12	3	6	H	8S		2	18.5	2	23
13	3	9	L	1S [c]		2	9.9	0	—
14	3	9	M	6S	11S	4	16.5	4	18
15	3	9	H	9N		2	16.6	2	19
16	3	12	L	2N [c]		2	11.7	0	—
17	3	12	M	4S [c]		2	16.4	0	—
18	3	12	H	7N	12N	4	17.5	4	16.5

(*b*) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.1)	*P*	STD. DEV.	CV
Total	526.675	47						
Between designs	229.685	17	13.511	1.36	1.72	NS		
Within designs	296.990	30	9.900				3.15	20%

(*c*) ANALYSIS OF VARIANCE, DEFLECTIONS (BENKELMAN BEAM)

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.1)	*P*	STD. DEV.	CV
Total	229.875	31						
Between designs	121.875	11	11.080	2.05	1.91	<0.1		
Within designs	108.000	20	5.400				2.32	13%

[a] Constants: D_3 (= 12 in.), S_1, S_2, S_4. Section length = 560 ft.
[b] L, M, H = low, medium, high, based on percentage of limestone added to natural subgrade or embankment material and a Florida strength test.
[c] Benkelman beam data not available.
[d] Axle load at 20,000 lb.

TABLE B-2

FLORIDA, MARIANNA PROJECT (UNIFORM SECTIONS)

(*a*) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_2	D_3	S_2[b]	SECTION NO.	DYNAFLECT DEFLECTION (0.001 IN.) NO. OBS.	MEAN
1	4	28	L	11	2	12.2
2	4	28	H	3	2	11.9
3	6	26	L	13	2	10.5
4	6	26	H	5	2	11.9
5	8	24	L	16	2	13.1
6	8	24	H	8	2	11.4

(*b*) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.1)$	P	STD. DEV.	CV
Total	40.282	11						
Between designs	7.487	5	1.497	0.27	3.11	NS		
Within designs	32.795	6	5.466				2.34	20%

[a] Constants: D_1, $D_2 + D_3$ (= 32 in.), S_1, S_3, S_4. Section length = 100 ft.
[b] L, H = low, high, based on Hubbard-Field stability test for hot-mix sand-asphalt.

TABLE B-3

FLORIDA, MARIANNA PROJECT (TAPERED SECTIONS)

(*a*) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_2	D_3	S_2[b]	SECTION NO. AND TEST POINT[c] REP. 1	REP. 2	DYNAFLECT DEFLECTION (0.001 IN.) NO. OBS.	MEAN
1	4.7	27.3	L	10B	12B	2	11.4
2	4.7	27.3	H	2B	4B	2	12.0
3	5.3	26.7	L	10A	12A	2	11.6
4	5.3	26.7	H	2A	4A	2	11.5
5	6.7	25.3	L	9B	14B	2	10.7
6	6.7	25.3	H	1B	6B	2	12.8
7	7.3	24.7	L	9A	14A	2	9.8
8	7.3	24.7	H	1A	6A	2	12.1

(*b*) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.10)$	P	STD. DEV.	CV
Total	25.739	15						
Between designs	12.094	7	1.728	1.01	2.62	NS		
Within designs	13.645	8	1.706				1.31	11%

[a] Constants: D_1, $D_2 + D_3$ (= 32 in.), S_1, S_3, S_4. Section length = 100 ft.
[b] L, H = low, high, based on Hubbard-Field stability test for hot-mix sand-asphalt.
[c] Test point A located 33.3 ft from thick end of tapered base course; test point B located 66.7 ft from thick end of tapered base course.

TABLE B-4

ALABAMA

(a) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_1	D_2	D_3	D_4	S_1	S_2[b]	S_3[b]	S_4[b]	S_5[b]	SECTION NO.	DYNAFLECT DEFLECTION (0.001 IN.) NO. OBS.	MEAN
1	2	5	5	6	Unknown	182	137	25	10	63-65-3(1)	15	14.6
2	1.5	6	5	6	Unknown	78	19	40	10	4-55-2(1)	15	26.3
3	0.6	6	6	—	Unknown	60	25	—	12	4-24C-1(1)	15	29.9
4	5.5	10	6	—	Unknown	100+	30	—	22	11-65I-3(1)	15	13.6

(b) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.01)$	P	STD. DEV.	CV
Total	4201.462	59						
Between designs	3053.143	3	1017.714	49.63	4.31	<0.01		
Within designs	1148.319	56	20.506				4.53	22%

[a] Section length = 2,500 ft.
[b] Alabama CBR.

TABLE B-5

MISSOURI

(a) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_1	D_2	D_3	SECTION NO. REP. 1	REP. 2	DYNAFLECT DEFLECTION (0.001 IN.) NO. OBS.	MEAN
1	3.25	2	2	19	16	10	35.6
2	3.25	2	4	22	17	10	27.3
3	3.25	4	2	20		5	34.4
4	3.25	4	4	24	18	12	21.1
5	4.00	0	8	23	21	12	20.0

(b) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	$F(0.01)$	P	STD. DEV.	CV
Total	3442.336	48						
Between designs	2026.355	4	506.589	15.74	3.83	<0.01		
Within designs	1415.981	44	32.181				5.67	21%

[a] Constants: S_1, S_2, S_3, S_4. Section length = 300 ft.

TABLE B-6

ILLINOIS

(a) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_2	D_3	S_2[b]	SECTION NO.	DYNAFLECT DEFLECTION (0.001 IN.) NO. OBS.	MEAN
1	8.5	23	Cr. stone	3	12	36.3
2	12	4	CAM	4	4	18.4
3	10	4	BAM	5	4	21.3

(b) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	F(0.01)	P	STD. DEV.	CV
Total	1594.877	19						
Between designs	1330.228	2	665.114	42.7	6.36	<0.01		
Within designs	264.649	17	15.568				3.95	13%

[a] Constants: D_1 (= 4.5 in.), S_1, S_3, S_4. Section length = 890 to 2,550 ft.
[b] Type of material: CAM = cement-aggregate mixture; BAM = bituminous-aggregate mixture.

TABLE B-7

MINNESOTA

(a) DESIGN AND DEFLECTION DATA

DESIGN NO.	DESIGN VARIABLES[a] D_1	D_2	D_3	S_1, S_2, S_3	S_4[b]	SECTION NO.	DEFLECTION (0.001 IN.) DYNAFLECT NO. OBS.	MEAN	STD. DEV.	CV (%)	BENKELMAN BEAM NO. OBS.	MEAN	STD. DEV.	CV (%)
1	7	9	12	c	45	23	11	13.6	1.3	10	11	13	1.8	14
2	4.5	6.5	0	c	67	18	11	13.7	1.5	11	11	13	1.9	15
3	4	7.5	6	c	28	20	11	31.7	2.2	7	11	22	2.5	11
4	5.5	6	12.5	c	17	21	11	27.5	3.8	14	11	20	4.6	23
5	7	6	16	c	15	22	11	22.5	1.5	7	11	19	3.1	16
6	4	3.5	6	c	9	35	11	77.2	10.1	13	11	68	6.7	10
7	6	0	8	c	23	47	11	50.5	5.3	10	11	57	6.9	12
8	3	3	11	c	28	24	11	57.0	6.5	11	11	57	3.6	6
9	c	c	c	c	c	A	11	64.0	8.7	14	11	61	9.7	16
10	c	c	c	c	c	B	11	73.1	13.1	18	11	78	20.6	26
						Avg.		43		12		41		15

(b) ANALYSIS OF VARIANCE, DEFLECTIONS (DYNAFLECT)

SOURCE OF VARIATION	SS	DF	MS	F	F(0.01)	P	STD. DEV.	CV
Total	62522.615	109						
Between designs	58089.125	9	6454.35	145.6	2.72	<0.01		
Within designs	4433.490	100	44.33				6.66	16%

(c) ANALYSIS OF VARIANCE, DEFLECTIONS (BENKELMAN BEAM)

SOURCE OF VARIATION	SS	DF	MS	F	F(0.01)	P	STD. DEV.	CV
Total	71908.691	109						
Between designs	65221.419	9	7246.82	108.4	2.72	<0.01		
Within designs	6687.272	100	66.87				8.18	20%

[a] Section length = 500 ft.
[b] R value.
[c] Unknown, but believed to vary between designs.

TABLE B-8

ALABAMA

(*a*) SERVICEABILITY INDEX DATA [a]

DESIGN NO.	SUBSECTION NO.[b] REP. 1	SUBSECTION NO.[b] REP. 2	SERV. INDEX [c] NO. OBS.	SERV. INDEX [c] MEAN
1	63-6S-3(1)A	63-6S-3(1)B	2	4.05
2	4-5S-2(1)A	4-5S-2(1)B	2	3.25
3	4-24C-1(1)A	4-24C-1(1)B	2	2.65
4	11-65I-3(1)A	11-65I-3(1)B	2	4.10

(*b*) ANALYSIS OF VARIANCE, SERVICEABILITY INDEX [d]

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.01)	*P*	STD. DEV.	CV
Total	2.909	7						
Between designs	2.894	3	0.9647	254	16.7	<0.01		
Within designs	0.015	4	0.0038				0.062	2%

[a] See Table B-4 for design variables.
[b] Subsection length = 1,200 ft.
[c] From average SV, Texas CHLOE and roughometer.
[d] Texas CHLOE and roughometer.

TABLE B-9

MISSOURI

(*a*) SERVICEABILITY INDEX DATA [a]

DESIGN NO.	SECTION NO. REP. 1	SECTION NO. REP. 2	SERV. INDEX [b] NO. OBS.	SERV. INDEX [b] MEAN
1	19	16	2	4.70
2	22	17	2	4.50
3	20		1	4.40
4	24	18	2	4.65
5	23	21	2	4.80

(*b*) ANALYSIS OF VARIANCE, SERVICEABILITY INDEX [b]

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.10)	*P*	STD. DEV.	CV
Total	0.3800	8						
Between designs	0.1550	4	0.03875	0.689	4.11	NS		
Within designs	0.2250	4	0.05625				0.237	5%

[a] See Table B-5 for design variables.
[b] Texas CHLOE.

TABLE B-10

ILLINOIS

(*a*) SERVICEABILITY INDEX DATA [a]

DESIGN NO.	SUBSECTION NO.[b] REP. 1	REP. 2	REP. 3	REP. 4	REP. 5	SERV. INDEX [c] NO. OBS.	SERV. INDEX [c] MEAN
1	3A	3B	3C	3D	3E	5	4.12
2	4A	4B				2	4.05
3	5A	5B				2	4.10

(*b*) ANALYSIS OF VARIANCE, SERVICEABILITY INDEX [c]

SOURCE OF VARIATION	SS	DF	MS	*F*	*F*(0.10)	*P*	STD. DEV.	CV
Total	0.88000	8						
Between sections	0.00700	2	0.00350	0.0241	3.46	NS		
Within sections	0.87300	6	0.14550				0.381	9%

[a] See Table B-6 for design variables.
[b] Subsection length = 390 to 550 ft.
[c] Texas CHLOE.

TABLE B-11

FLORIDA, CHIEFLAND PROJECT

(*a*) COMPARISON OF TEXAS AND FLORIDA PROFILOMETERS

	PROFILOMETER SLOPE VARIANCE [a]			
	INNER WHEEL PATH		OUTER WHEEL PATH	
SEC. NO.	TEXAS	FLORIDA	TEXAS	FLORIDA
5S	3.8	5.5	5.3	6.4
5N	3.5	4.8	9.0	4.4
6S	3.0	6.3	3.6	3.5
6N	3.7	4.3	4.2	3.7
7S	11.7	7.0	5.8	5.7
7N	2.5	3.9	3.0	4.5
8S	3.1	2.8	3.9	5.2
8N	5.7	6.2	4.7	6.4
Avg.	4.6	5.1	4.9	5.0

(*b*) ANALYSIS OF VARIANCE, SLOPE VARIANCE [a]

SOURCE OF VARIATION	SS	DF	MS	*F*	REQUIRED *F*	*P*
Total	111.947	31	3.611			
Between sections	51.965	7	7.424	2.07	$F(0.10) = 2.78$	NS
Between wheel paths	0.070	1	0.070	0.195	$F(0.10) = 3.59$	NS
Residual (a)	25.067	7	3.581			
Between CHLOEs	0.525	1	0.525	0.217	$F(0.10) = 3.18$	NS
CHLOE × W.P.	0.383	1	0.383			
Residual (b)	33.937	14	2.424			

[a] Uncorrected.

TABLE B-12

MISSOURI

(*a*) COMPARISON OF TEXAS AND MISSOURI PROFILOMETERS

	PROFILOMETER SLOPE VARIANCE [a]			
	INNER WHEEL PATH		OUTER WHEEL PATH	
SEC. NO.	TEXAS	MISSOURI	TEXAS	MISSOURI
24	2.9	2.9	4.9	5.9
23	2.7	2.9	4.3	4.9
22	3.8	4.1	4.5	4.7
21	3.3	3.5	3.9	4.2
20	4.0	4.0	5.4	6.1
19	3.6	4.1	5.3	5.6
18	2.8	3.1	4.5	4.1
17	3.6	4.0	6.0	6.8
16	3.2	3.7	4.0	4.5
Avg.	3.3	3.6	4.8	5.2

(*b*) ANALYSIS OF VARIANCE, SLOPE VARIANCE [a]

SOURCE OF VARIATION	SS	DF	MS	*F*	REQUIRED *F*	*P*
Total	36.810	35	1.052			
Between sections	9.610	8	1.201	2.22	$F(0.10) = 2.59$	NS
Between wheel paths	20.854	1	20.854	38.55	$F(0.10) = 11.3$	<0.01
Residual (a)	4.326	8	0.541			
Between CHLOEs	1.138	1	1.138	22.31	$F(0.01) = 8.68$	<0.01
CHLOE × W.P.	0.071	1	0.071			
Residual (b)	0.811	16	0.051			

[a] Uncorrected.

APPENDIX C

SEISMIC TESTS

In recent years seismic methods of testing materials have received considerable attention. It has been observed that the velocity of propagation of the compressional wave alone provides some information about the physical characteristics of a given material. If the velocity of the shear wave also is known, the material's elastic constants may be calculated (provided, of course, the material deforms in accordance with Hooke's law).

In this work primary importance was attached to determining the compressional velocity of each of the materials used in the construction of the test facility. (The more difficult problem of determining the shear velocities of the materials was also attempted, but the results obtained were not conclusive.)

Two methods were used to measure the seismic velocities. A brief discussion of each method follows.

One of the methods (*11*) is generally referred to as a "pulse" technique or "ultrasonic" technique. The apparatus (Fig. C-1), assembled at Texas Transportation Institute, consisted of four major components: a pulse generator, a source transducer, a receiver transducer, and an oscilloscope. The pulse generator actuates the source transducer (a ceramic piezoelectric crystal) and triggers the horizontal sweep of the oscilloscope. Actuation of the source transducer causes seismic energy to be transmitted through the sample to the receiver transducer (another ceramic piezoelectric crystal). The output of the receiver transducer is displayed on the horizontal sweep of the oscilloscope. A time delay device built into the oscilloscope is used to make the arrival of the seismic energy at the receiver transducer coincide with the start of the horizontal sweep. The time required for the seismic energy to travel through the sample may then be read directly from the time delay device. Knowing the length of the sample, the seismic velocity may be determined.

This method is generally used to measure seismic velocities in cores or prepared samples. However, its use was extended in this work to testing the material in situ as well. This was accomplished by digging small holes, in which were placed the source and receiver transducers, to measure the velocity along a path parallel to the road surface.

In order to have an independent method of measuring velocities, so that a comparison could be made with the values obtained from the pulse technique, measurements were made with a multi-channel portable seismograph.* This unit is designed for shallow refraction studies which might be encountered in foundation design, groundwater studies, and other problems where the layers of interest are several feet or tens of feet thick.

Measurements were made with the seismograph on each section, as each component of the test facility was completed. The procedure was to lay out six geophones at intervals of 5 ft along a line parallel to the long axis of the section (Fig. C-2) with which to record the seismic energy from the source, which was a hammer blow using a steel plate as a coupler. The seismograph provided a Polaroid picture, from which the time required for the energy to travel from the source to each geophone could be read. Then, knowing the distance to each geophone, and assuming that the energy traveled directly from source to geophone, the velocity of the wave front could be determined at each of the six geophones. However, in practice these values were averaged by taking the velocity to be the inverse of the slope of the best fit line drawn through the points on a graph where time of travel is plotted as ordinates against distance from the source (Fig. C-3).

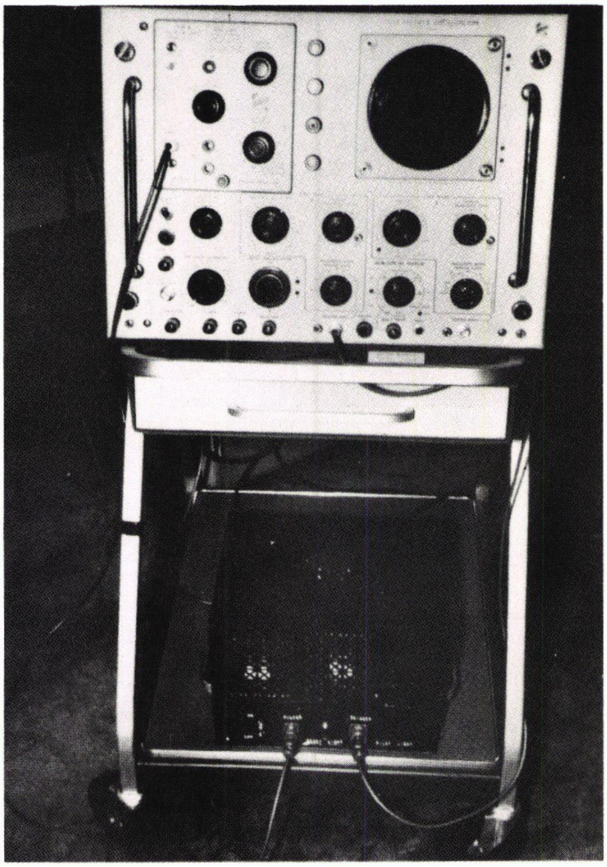


Figure C-1. Oscilloscope and pulse generator on mobile cart.

* Geo Space GT-2 Portable Seismograph, Geo Space Corp., Houston, Tex.

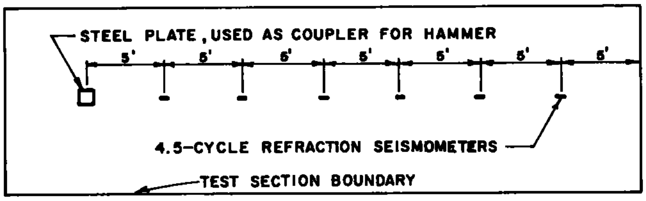


Figure C-2. Seismometer arrangement used to determine compressional wave velocity.

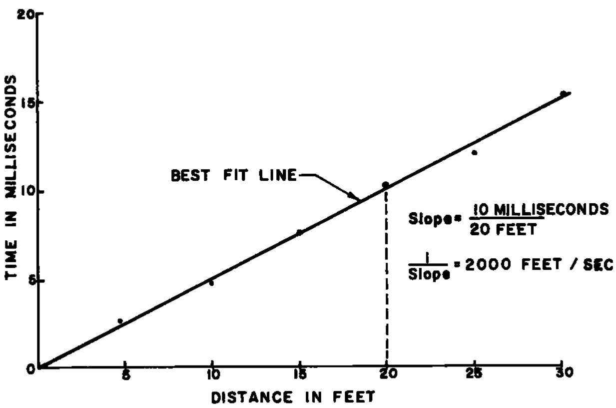


Figure C-3. Travel time plot for energy traveling direct path from source to geophones.

The seismometers, designed to measure motion in a plane horizontal to the earth's surface, were oriented such that they would measure motion in the direction of propagation of the seismic energy. This orientation was chosen because elastic theory predicts that this should be the direction of particle motion for a compression wave.

Table C-1 gives a summary of the results of the compressional velocity measurements made by both the portable seismograph and the pulse technique. In situ measurements were made during construction at a time when the lime- and cement-stabilized materials had not completely cured. Thus the velocities measured in these materials were probably less than the ultimate attained after curing.

The velocities determined by the two methods for the same material do not show the agreement that was expected. This lack of agreement is too great to be attributed to experimental error in either method; therefore, one must seek an explanation in the basic assumptions of the methods. Before considering the assumption of both methods it should be mentioned that although the velocities obtained by the two methods show poor agreement, the velocities were consistent in the sense that for a given material the velocity determined by the pulse technique was always greater than that determined by the portable refraction device. Any satisfactory explanation must be in accord with this fact.

The use of the pulse technique to determine compressional velocity has been well founded by many previous investigators—Leslie (*12*), Birch (*13*), and others. This, together with the fact that no plausible reason could be found why this method would not give the correct values, caused acceptance of these values as being close to the true compressional wave velocities. Thus, to explain the discrepancy between the velocities obtained by the two methods, the measurements made with the portable refraction seismograph must be considered.

The measurements made with the portable refraction device were made on the top of each component as the test facility was constructed. Because the tests were made on the top of a component just completed, it was assumed

TABLE C-1

COMPRESSIONAL VELOCITIES BY TWO METHODS

			PULSE TECHNIQUE						PORTABLE SEISMOGRAPH		
MATERIAL INDEX	WHERE USED	DESCRIPTION	LAB. VEL.[a] (FPS)	NO. OBS.	STD. DEV.	FIELD VEL.[b] (FPS)	NO. OBS.	STD. DEV.	FIELD VEL.[b] (FPS)	NO. OBS.	STD. DEV.
0	Foundation	Plastic clay, undisturbed	1744	5	304	—	—	—	1180	7	81
1	Embankment	Plastic clay	3492	2	430	2412	12	274	1460	10	39
2	Embankment	Sandy clay	2370	3	265	2576	24	523	1435	20	98
3	Embankment	Sandy gravey	—	—	—	3721	21	1423	1714	11	190
4	Base & subb	Cr. limestone	—	—	—	5222	3	574	2920	12	305
5	Base & subb	Cr. limestone +2% lime	—	—	—	5448	2	16	2833	19	543
6	Base & subb	Cr. limestone +4% cement	—	—	—	7309	3	701	5436	22	1074
7	Surfacing	Asph. conc.	8463	8	208	—	—	—	(See Table C-2)		

[a] Tests made on cores obtained during construction; asphaltic concrete core tested at 70 F.
[b] Tests made during construction; stabilized materials had not completely cured.

TABLE C-2

COMPRESSIONAL VELOCITIES MEASURED ON TOP OF ASPHALT SURFACE LAYER

SECTION NO.	MATERIAL		AVG. VEL. (FPS)	NO. OBS.	STD. DEV.
	BASE	SUBBASE			
9-12	Cr. limestone	Cr. limestone	2990	4	307
17-23, 28-29	Cr. limestone +2% lime	Cr. limestone +2% lime	3480	7	202
5-8	Cr. limestone	Cr. limestone +4% cement	4928	4	1210
1-4	Cr. limestone +4% cement	Cr. limestone	6615	4	778
13-16	Cr. limestone +4% cement	Cr. limestone +4% cement	7513	4	920

that the velocity measured would be the compressional velocity of the material in that particular component, if the underlying materials exhibited lesser velocities.

The average compressional velocity of each of the materials determined by this procedure is given in Table C-1; however, measurements made on top of the base in Sections 5, 6, 7, 8, 24, and 27 were excluded because, according to Table 3, the stiffness of the base materials in these sections was judged to be less than that of the subbase materials beneath them. Also, the measurements made on the top of the asphalt are not included, but are given separately in Table C-2, inasmuch as the velocity measured appears to be primarily influenced by the material of the base and subbase.

In Table C-2 the velocities measured on the top of the asphalt are grouped according to the material of the base and subbase. In each of the groups the average surface thickness is 3 in. and the average base and subbase thickness is 8 in.; therefore, the differences in the average velocities are apparently due to the base and subbase materials underlying the asphaltic concrete.

The assumption made in determining the velocities was tantamount to assuming that "ray theory" seismology was valid. In ray theory seismology, the rays (or perpendiculars to the wave fronts) are assumed to obey Snell's law; from this one is able to make predictions concerning the path the seismic energy will take if the elastic constants of the medium are known. The theory of refraction seismology, as used in studies of the earth (considering the earth as a layered medium), is based on a particular ray of an infinite number of rays. This ray is called the "critical angle ray," and is defined by

$$\sin i_c = V_1/V_2 \qquad \text{(C-1)}$$

in which V_1 and V_2 are the compressional wave velocities of layers 1 and 2, respectively (Fig. C-4), i_c is the critical angle, and $V_2 > V_1$. Figure C-4 shows the path the energy appears to follow for this special case, as well as the path for the direct wave. Figure C-5 is a theoretical travel time plot (where the time of travel for the first arriving energy is plotted against the distance from source to receiver) for the layering of Figure C-4. The travel time plot shows that from the source out to some particular distance, X_c (called the critical distance), the direct wave arrives first, but past that distance the refracted wave arrives first even though it travels a greater distance. The reason for this is that it travels at a greater velocity (V_2) through the segment (BC) of its path. The velocity of each layer is equal to the inverse of the slope of the corresponding segment of the travel time plot.

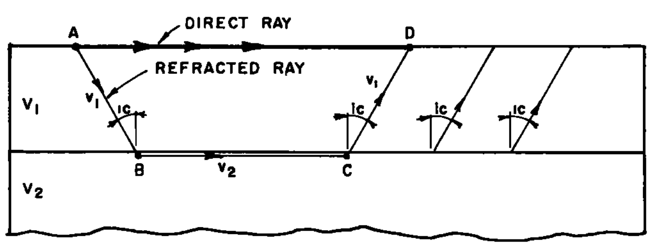


Figure C-4. Ray path for critical-angle ray and direct ray.

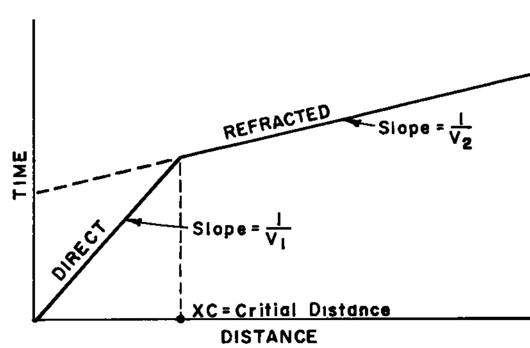


Figure C-5. Theoretical travel time plot for two-layer critical-angle refraction.

The critical angle refraction technique has been widely used in earth studies, and has provided much useful information. An important characteristic of this method becomes apparent if Eq. C-1 is considered in the light of having a velocity V_2 less than V_1. This results in the sine of i_c being greater than unity, an impossibility. Therefore, there will be no critical angle for a layer with a velocity less than any layer above it, and this method will provide no information about such a layer. Thus, if ray theory were valid and the material on which the tests were made had a greater velocity than any material beneath it, the velocity measured should be that of the material on which the test was made. The results of the experiment have caused doubt as to the validity of this assumption. It now appears likely that ray theory was not valid and that this was the cause of the poor agreement of the two methods. It is believed that ray theory was inadequate for this study because the wavelength of the first recorded energy at each geophone was much larger than the thickness of the components in the test facility.

An examination of the records from the portable seismograph showed that a value of 0.005 sec would be a reasonable minimum value for the period (t) of the first arriving energy. If one considers a velocity of 1,000 ft/sec, which is very low, and assumes

$$\lambda = v\,t \qquad \text{(C-2)}$$

to be valid, this would predict a wavelength of 5 ft as a minimum. It appears likely that having a wavelength much larger than the thickness of the components would result in the wave traveling in the material below the component as well as in the component itself. This would cause any measured velocity in Table C-1 to be less than the actual velocity of the component on which the measurements were made, a result that satisfies the criteria mentioned previously concerning the relation between the velocity determined by the pulse technique and that determined by the portable refraction seismograph.

From the results of this work it seems unlikely that a "typical" portable refraction device designed for general engineering purposes would be an adequate tool for determining directly the compressional wave velocities in individual structural components of flexible pavements, because such an instrument is not designed to initiate and detect the high-frequency, short-wavelength energy apparently required for this application. However, Phelps and Cantor (*14*) reported using successfully the seismic-refraction technique to determine the velocity of thin layers of asphalt overlying concrete. They used a recording system which was much more sensitive to higher frequencies, much shorter intervals between the energy source and detectors, and a less energetic source. With this specialized instrumentation, the seismic refraction technique might be a useful tool for testing highway construction materials.

APPENDIX D

MULTIPLE-ERROR REGRESSION TECHNIQUE

In the classical least-squares method of fitting a linear model to data collected in an experiment involving several variables, it is assumed that the values of all but one—the dependent or response variable—are known precisely. Frequently, however, there are errors of measurement in all the variables, and when this is the case the classical method yields a biased estimate of the regression coefficients. Inasmuch as the objective of most experiments is to obtain unbiased estimates of these coefficients, it is apparent that measurement errors in the independent variables should not be ignored.

The regression technique described herein accounts for errors in all variables. It is essentially the same as a method described by Johnston (*15*), but includes a new concept—that of the "quality" of a variable. Because of the introduction of this concept, and because it was desired to confirm Johnston's results by independent means, it was necessary to perform the mathematical operations described in the following sections.

The reader who does not desire to follow the derivations will find the essential elements of the method beginning with the section on "Quality of a Variable."

ASSUMPTIONS

Let it be supposed that an experiment involves a set of p variables, the true values of which are known to be linearly dependent. The variables are designated $X_1, X_2, \ldots, X_j, \ldots, X_p$.

In the course of the experiment the whole set of variables is measured from time to time (or from place to place, depending on the nature of the experiment). At one of these times (or places) the ith set of measurements, $X_{i1}, X_{i2}, \ldots, X_{ij}, \ldots, X_{ip}$, is obtained.

Corresponding to the ith set of measurements, there is a set of true values, $\tilde{X}_{i1}, \tilde{X}_{i2}, \ldots, \tilde{X}_{ij}, \ldots, \tilde{X}_{ip}$, and a set of measurement errors, $e_{i1}, e_{i2}, \ldots, e_{ij}, \ldots, e_{ip}$.

It is assumed that the measurement errors are random, independent, and normally distributed, with a mean value of zero. The measurement error, e_{ij}, is defined by

$$X_{ij} = \tilde{X}_{ij} + e_{ij} \quad \text{(D-1)}$$

The true values of the variables, according to the assumption of linear dependence, satisfy

$$A_0 + \sum_{j=1}^{p} A_j \tilde{X}_{ij} = 0 \quad \text{(D-2)}$$

in which $A_0, A_1, \ldots, A_p$ are constants.

DERIVATION OF EXPRESSION TO BE MINIMIZED

Eqs. D-1 and D-2 lead directly to a relationship involving the measured values and measurement errors, as follows:

$$A_0 + \sum_{j=1}^{p} A_j X_{ij} = \sum_{j=1}^{p} A_j e_{ij} \quad \text{(D-3)}$$

Squaring Eq. D-3 and summing over the index i gives

$$\sum_{i=1}^{n} \left(A_0 + \sum_{j=1}^{p} A_j X_{ij}\right)^2 = \sum_{i=1}^{n} \left(\sum_{j=1}^{p} A_j e_{ij}\right)^2 \quad \text{(D-4)}$$

in which n is the total number of times the set of variables has been measured.

Eq. D-4 may be simplified somewhat by eliminating A_0, as indicated in the following.

The error term is independent of A_0, so Eq. D-4 is differentiated with respect to A_0, and the resulting equation is solved for A_0, giving

$$A_0 = -\frac{1}{n} \sum_{i=1}^{n} \sum_{j=1}^{p} A_j X_{ij} \quad \text{(D-5a)}$$

or, more briefly,

$$A_0 = -\sum_{j=1}^{p} A_j \bar{X}_j \quad \text{(D-5b)}$$

in which $\bar{X}_j$ is the mean of the n measured values of the variable X_j.

Substitution of the right side of Eq. D-5b in Eq. D-4 gives an equation in terms of the deviations of the measured variables from their means:

$$\sum_{i=1}^{n} \left(\sum_{j=1}^{p} A_j V_{ij}\right)^2 = \sum_{i=1}^{n} \left(\sum_{j=1}^{p} A_j e_{ij}\right)^2 \quad \text{(D-6)}$$

in which $V_{ij} = X_{ij} - \bar{X}_j =$ the deviation of X_{ij} from the mean of the n measured values of X_j.

The right side of Eq. D-6 can be expanded into the sum of a series of terms of the type $A_j A_k \sum_{i=1}^{n} e_{ij} e_{ik}$. But, for large n, every term for which $j \neq k$ has an expected value of zero as a consequence of the previously stated assumption regarding the measurement errors, e_{ij}. If terms for which $j \neq k$ are neglected, there will remain in the series only terms of the type $A_j^2 \sum_{i=1}^{n} e_{ij}^2$ $(j = 1, \ldots, p)$. Thus, Eq. D-6 may be written in the form

$$\sum_{i=1}^{n} \left(\sum_{j=1}^{p} A_j V_{ij}\right)^2 = A_1^2 \sum_{i=1}^{n} e_{i1}^2 + \ldots + A_p^2 \sum_{i=1}^{n} e_{ip}^2 \quad \text{(D-7)}$$

Without loss of generality, each constant, A_j, is separated into two arbitrary factors, C_j and M_j, and one of the factors is defined as

$$A_j \equiv C_j M_j \quad \text{(D-8)}$$

in which

$$M_j \equiv \left[\sum_{i=1}^{n} e_{ip}^2 \Big/ \sum_{i=1}^{n} e_{ij}^2\right]^{1/2} \quad \text{(D-9)}$$

Also introduced is a new variable, Z_{ij}, defined by

$$Z_{ij} \equiv M_j V_{ij} \quad \text{(D-10)}$$

From Eqs. D-8 and D-10 it can be seen that

$$A_j V_{ij} = C_j M_j V_{ij} = C_j Z_{ij} \quad \text{(D-11)}$$

and from Eqs. D-8 and D-9 it is clear that

$$A_j^2 \sum_{i=1}^{n} e_{ij}^2 = C_j^2 M_j^2 \sum_{i=1}^{n} e_{ij}^2 = C_j^2 \sum_{i=1}^{n} e_{ip}^2 \quad \text{(D-12)}$$

In Eq. D-7 the following substitutions are now made:

For	Substitute	Basis
$A_j V_{ij}$	$C_j Z_{ij}$	Eq. D-11
$A_j^2 \sum_{i=1}^{n} e_{ij}^2$	$C_j^2 \sum_{i=1}^{n} e_{ip}^2$	Eq. D-12

giving

$$\sum_{i=1}^{n} \left(\sum_{j=1}^{p} C_j Z_{ij}\right)^2 = \left(\sum_{j=1}^{p} C_j^2\right)\left(\sum_{i=1}^{n} e_{ip}^2\right) \quad \text{(D-13)}$$

which can be rewritten as

$$\sum_{i=1}^{n} \left[\frac{C_1 Z_{i1} + \ldots + C_p Z_{ip}}{(C_1^2 + \ldots + C_p^2)^{1/2}}\right]^2 = \sum_{i=1}^{n} e_{ip}^2 \quad \text{(D-14)}$$

(It is of interest to note that if $p = 3$, and the quantities Z_{i1}, Z_{i2} and Z_{i3} are regarded as the rectangular coordinates of a point in three-dimensional space, the expression enclosed in brackets represents the perpendicular distance from the point Z_{i1}, Z_{i2}, Z_{i3} to the plane $C_1 Z_1 + C_2 Z_2 + C_3 Z_3 = 0$.)

In the interest of further simplification, the constant B_j is defined by

$$B_j \equiv \frac{C_j}{(C_1^2 + \ldots + C_p^2)^{1/2}} \quad \text{(D-15)}$$

and Eq. D-14 is written, in terms of the new set of constants, as

$$\sum_{i=1}^{n} (B_1 Z_{i1} + \ldots + B_p Z_{ip})^2 = \sum_{i=1}^{n} e_{ip}^2 \quad \text{(D-16)}$$

From the definition of B_j (Eq. D-15) it is clear that

$$B_1^2 + B_2^2 + \ldots + B_p^2 - 1 = 0 \quad \text{(D-17)}$$

Values of the coefficients $B_1, \ldots, B_p$ can be estimated by minimizing the left side of Eq. D-16, subject to the constraint expressed by Eq. D-17. From them, estimates of the coefficients $A_0, A_1, \ldots, A_p$ can be computed as shown in the next section.

PROCEDURE FOR ESTIMATING THE COEFFICIENTS B_j AND A_j

To minimize Eq. D-16, subject to Eq. D-17, use is made of "Lagrange's method of multipliers" (*16*), as follows:

Let

$$\alpha = \sum_{i=1}^{n} (B_1 Z_{i1} + \ldots + B_p Z_{ip})^2 \qquad \text{(D-18)}$$

$$\beta = B_1^2 + \ldots + B_p^2 - 1 = 0 \qquad \text{(D-19)}$$

and

$-\lambda =$ the Lagrange multiplier.

According to the Lagrange technique, α will have an extreme value when the $p + 1$ parameters ($\lambda, B_1, B_2, \ldots, B_p$) have values determined by the following $p + 1$ equations:

$$\left.\begin{aligned} \frac{\partial\alpha}{\partial B_1} - \lambda\frac{\partial\beta}{\partial B_1} &= 0 \\ \frac{\partial\alpha}{\partial B_2} - \lambda\frac{\partial\beta}{\partial B_2} &= 0 \\ &\cdot \\ &\cdot \\ &\cdot \\ \frac{\partial\alpha}{\partial B_p} - \lambda\frac{\partial\beta}{\partial B_p} &= 0 \end{aligned}\right\} \text{(D-20)}$$

Performing the indicated operations on Eqs. D-18 and D-19 forms a set of p linear equations corresponding to the p differential equations of Eqs. D-20 and the result is written as a matrix

$$\begin{bmatrix} W_{11} - \lambda & W_{12} & \ldots & W_{1p} \\ W_{21} & W_{22} - \lambda & \ldots & W_{2p} \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ \cdot & & & \cdot \\ W_{p1} & W_{p2} & \ldots & W_{pp} - \lambda \end{bmatrix} \times \begin{bmatrix} B_1 \\ B_2 \\ \cdot \\ \cdot \\ \cdot \\ B_p \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \end{bmatrix} \qquad \text{(D-21)}$$

in which

$$W_{jk} = \sum_{i=1}^{n} Z_{ij} Z_{ik} = W_{kj} \qquad \text{(D-22)}$$

Eq. D-21 has a nontrivial solution if (and only if) the determinant of the $p \times p$ matrix in Eq. D-21 is zero. This determinant can be made zero by choosing an appropriate value of λ. But because there are p values of λ that will make the determinant zero, it is necessary to choose the particular value that will result in minimizing $\sum_{i=1}^{n} e_{ip}^2$. The smallest positive value of λ is the root desired; Johnston (*15*) presents a proof of this statement.

It is also assumed that the reader is familiar with methods for finding the roots of the determinant of the symmetrical $p \times p$ matrix in Eq. D-21 (*17*).

Let $\hat{\lambda}$ be the smallest positive value of λ that will make the determinant zero. Substitute $\hat{\lambda}$ for λ in Eq. D-21, divide each equation (except the last) by B_p, and form $p - 1$ linear equations in matrix form, as follows:

$$\begin{bmatrix} W_{11} - \hat{\lambda} & W_{12} & \ldots & W_{1,p-1} \\ W_{21} & W_{22} - \lambda & \ldots & W_{2,p-1} \\ & & & \cdot \\ W_{p-1,1} & W_{p-1,2} & \ldots & W_{p-1,p-1} - \hat{\lambda} \end{bmatrix} \times \begin{bmatrix} B_1/B_p \\ B_2/B_p \\ \\ B_{p-1}/B_p \end{bmatrix} = \begin{bmatrix} -W_{1p} \\ -W_{2p} \\ \\ -W_{p-1,p} \end{bmatrix} \qquad \text{(D-23)}$$

These $p - 1$ equations can be solved for the $p - 1$ ratios, B_j/B_p ($j = 1, \ldots, p - 1$).

Now, according to Eq. D-15,

$$B_j/B_p = C_j/C_p \qquad \text{(D-24)}$$

and, according to Eq. D-8,

$$\frac{C_j}{C_p} = \frac{A_j/M_j}{A_p/M_p} \qquad \text{(D-25)}$$

from which

$$\frac{B_j}{B_p} = \frac{A_j/M_j}{A_p/M_p} \qquad \text{(D-26)}$$

It is noted from Eq. D-9 that $M_p = 1$, and (without loss of generality) it is assumed that $A_p = 1$. Substitution of 1 for M_p and A_p in Eq. D-26 gives

$$B_j/B_p = A_j/M_j \qquad \text{(D-27)}$$

Thus, the estimate, $\hat{A}_j$ of A_j is found from

$$\hat{A}_j = (B_j/B_p)M_j \qquad \text{(D-28)}$$

Eq. D-28 is the last step in the solution of the problem. Application of this regression technique presupposes some knowledge of measurement errors for each variable (see, for example, Eq. D-9). To make the technique somewhat easier to apply, the next section discusses the concept of the "quality" of a variable, and presents an alternate to Eq. 9 for computing the M_j.

QUALITY OF A VARIABLE

The quality, Q_j, of the variable, X_j, is defined as the ratio of the variance of X_j to the variance of the errors made in measuring X_i. The equivalent mathematical definition is the dimensionless ratio

$$Q_j \equiv \frac{\sum_{i=1}^{n} (X_{ij} - \bar{X}_j)^2}{\sum_{i=1}^{n} e_{ij}^2} \qquad \text{(D-29)}$$

It may be seen from Eq. D-29 that if the experiment is so designed that X_j varies widely about its mean, and if the errors made in measuring X_j are small, the quality of the variable is high. On the other hand, if X_j varies only slightly from its mean and the measurement errors are large, the quality is low.

From the foregoing it is clear that the quality of a variable depends not only on the precision with which it can be measured, but also on the design of the experiment.

USE OF THE QUALITY RATIOS

From Eqs. D-29 and D-9 it can be shown that the constant, M_j, can be defined in terms of the quality ratio, Q_j/Q_p, as follows:

$$M_j \equiv \left[\frac{Q_j}{Q_p} \times \frac{\sum_{i=1}^{n}(X_{ip}-\overline{X}_p)^2}{\sum_{i=1}^{n}(X_{ij}-\overline{X}_j)^2}\right]^{1/2} \qquad \text{(D-30)}$$

The unknown in Eq. D-30 is the quality ratio, Q_j/Q_p. To compute M_j (a necessary step if the multiple-error regression technique is to be used), the investigator must estimate this ratio. This may be difficult, but probably less so than estimating the ratio of the sums of the squared errors as required by Eq. D-9. Therefore, Eq. D-30, rather than Eq. D-9, was used for computing M_j in the analysis of Dynaflect data described in this report.

APPLICATION

This section describes how the equations derived in the preceding sections may be used in estimating the constants in the regression model.

The model is

$$A_0 + A_1X_1 + A_2X_2 + \ldots + A_jX_j + \ldots + A_{p-1}X_{p-1} + X_p = 0 \qquad \text{(D-31)}$$

Steps to be followed in estimating the regression coefficients, A_j, are given in the following sequence.

1. To each variable, X_j, assign a quality ratio, Q_j/Q_p, in which Q_j is defined as in Eq. D-29.

2. Compute p values of $\sum_{i=1}^{n} V_{ij}^2$ $(j = 1, \ldots, p)$ from

$$\sum_{i=1}^{n} V_{ij}^2 = \sum_{i=1}^{n}(X_{ij}-\overline{X}_j)^2 \qquad \text{(D-32)}$$

3. Compute p values of M_j $(j = 1, \ldots, p)$ from

$$M_j \equiv \left[\frac{Q_j}{Q_p} \times \frac{\sum_{i=1}^{n}(X_{ip}-\overline{X}_p)^2}{\sum_{i=1}^{n}(X_{ij}-\overline{X}_j)^2}\right]^{1/2} \qquad \text{(D-33)}$$

4. Compute the p^2 elements of the symmetrical determinant,

$$\begin{vmatrix} W_{11} & W_{12} \ldots & W_{1p} \\ W_{21} & W_{22} \ldots & W_{2p} \\ & & \vdots \\ W_{p1} & W_{p2} \ldots & W_{pp} \end{vmatrix} \qquad \text{(D-34)}$$

from

$$W_{jk} = M_jM_k\sum_{i=1}^{n} V_{ij}V_{ik} = W_{kj} \qquad \text{(D-35)}$$

5. Find the least positive value, $\hat{\lambda}$, of λ that satisfies the determinantal equation

$$\begin{vmatrix} W_{11}-\lambda & W_{12} & \ldots & W_{1p} \\ W_{21} & W_{22}-\lambda & & W_{2p} \\ & & & \vdots \\ W_{p1} & W_{p2} & & W_{pp}-\lambda \end{vmatrix} = 0 \qquad \text{(D-36)}$$

(Note that this determinant is formed by subtracting λ from the diagonal elements of the determinant formed in Step 4).

6. Solve the following matrix equation for the $p-1$ ratios, B_j/B_p $(j = 1, \ldots, p-1)$:

$$\begin{bmatrix} W_{11}-\hat{\lambda} & W_{12} \ldots & W_{1,p-1} \\ W_{21} & W_{22}-\hat{\lambda} \ldots & W_{2,p-1} \\ & & \vdots \\ W_{p-1,1} & W_{p-1,2} \ldots & W_{p-1,p-1}-\hat{\lambda} \end{bmatrix} \times \begin{bmatrix} B_1/B_p \\ B_2/B_p \\ \vdots \\ B_{p-1}/B_p \end{bmatrix} = \begin{bmatrix} -W_{1p} \\ -W_{2p} \\ \vdots \\ -W_{p-1,p} \end{bmatrix} \qquad \text{(D-37)}$$

7. Find the estimates, $\hat{A}_j$, or the coefficients, A_j, $(j = 1, \ldots, p)$ from Eq. D-28.

8. Find the estimate, $\hat{A}_0$, of the intercept, A_0, from

$$\hat{A}_0 = -\sum_{j=1}^{p} \hat{A}_j\overline{X}_j \qquad \text{(D-38)}$$

If it is desired to force the regression plane through the origin (A_0 arbitrarily made zero), the procedure is the same as that given previously with the following two exceptions:

(a) Change Step 2 to read as follows:

Compute p values of $\sum_{i=1}^{n} V_{ij}^2$ $(j = 1, \ldots, p)$ from

$$\sum_{i=1}^{n} V_{ij}^2 = \sum_{i=1}^{n} X_{ij}^2 \qquad \text{(D-39)}$$

(b) Eliminate Step 8.

(Note that the value of M_j, computed in Step 3, is not affected by the change in the definition of V_{ij}, whereas the matrix element, W_{jk}, computed in Step 4, is affected.)

A NUMERICAL EXAMPLE

To illustrate the effect of variations in the quality ratios, Q_j/Q_p, on the regression coefficients, consider the following numerical example involving only two variables ($p = 2$), and hence only one quality ratio, Q_1/Q_2. The "data" (artificially contrived to emphasize certain features of the multiple-error technique), are given in Table D-1.

Using the multiple-error method, five analyses of the data were performed, each for a different quality ratio, Q_1/Q_2. The results are given in Table D-2 and are plotted, together with the data, in Figure D-1.

Comparisons with results given by the classical method can be made at extreme values of Q_1/Q_2. For example, if Q_1/Q_2 is made very small, as in analysis 1 of Table D-2, the coefficients given by the multiple-error method approach those computed by the classical procedure when

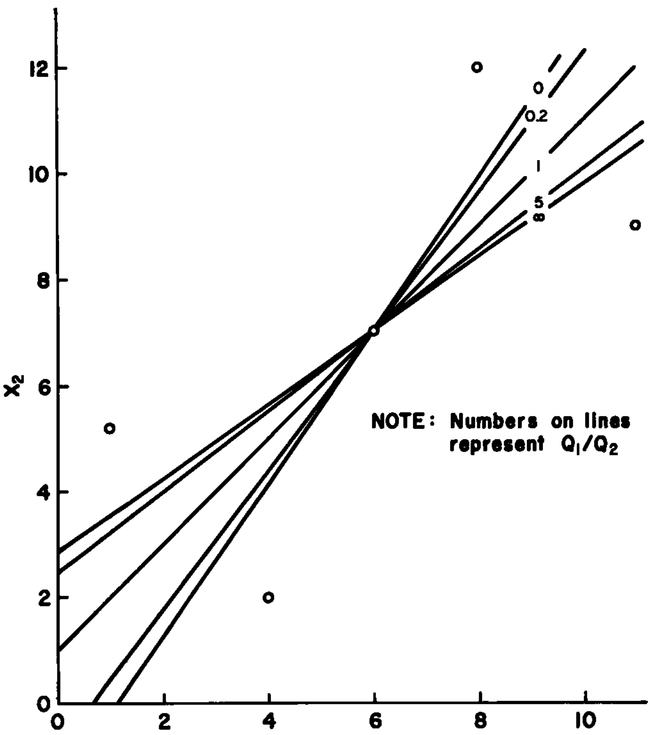


Figure D-1. Effect on the regression line of varying the quality ratio, Q_1/Q_2, in a two-variable analysis. The five circled points represent the data to which the model, $A_0 + A_1X_1 + X_2 = 0$, was fitted.

TABLE D-1

DATA FOR EXAMPLE

	MEASURED VALUES	
i	X_1	X_2
1	1	5
2	4	2
3	6	7
4	8	12
5	11	9

X_1 is regressed on X_2. If Q_1/Q_2 is made very large, as in analysis 5, the coefficients approach those given by the classical method when X_2 is regressed on X_1.

The result, clearly illustrated in Figure D-1, of making the quality of both variables the same ($Q_1/Q_2 = 1$), is a regression line that follows the visible trend of the data, and bisects the angle between the two lines obtained by the classical method. Also apparent from the figure is the fact that all possible regression lines lie between the extremes given by the classical method.

This two-variable example hopefully will confirm for the reader certain conclusions reached by the writers regarding the multiple-error regression technique, as follows:

1. The multiple-error technique is general in the sense that it includes the classical method as a special case.
2. If measurement errors exist in more than one of the variables entering into an experiment, estimates of the regression coefficients made by the classical method will be biased. Resort to the multiple-error method (if estimates of the quality ratios can be made) may lead to better estimates of the coefficients.
3. Although the multiple-error method (not necessarily under that name) has been discussed in the literature, it has not, to the authors' knowledge, come into general use. However, it should.

TABLE D-2

EFFECT OF QUALITY RATIO ON ANALYSIS (MODEL: $A_0+A_1X_1+X_2=0$)

MULTIPLE ERROR METHOD				CLASSICAL METHOD			
ANALYSIS NO.	Q_1/Q_2	A_0	A_1	Q_1/Q_2	A_0	A_1	DEPENDENT VARIABLE
1	10^{-6}	1.70000	−1.44500	0	1.70000	−1.44500	X_1
2	0.20	0.87435	−1.31239	—	—	—	—
3	1.00	−1.00000	−1.00000	—	—	—	—
4	5.00	−2.42820	−0.76197	—	—	—	—
5	10^6	−2.86207	−0.68966	∞	−2.86207	−0.68966	X_2

APPENDIX E

OPERATIONS MANUAL FOR CHLOE PROFILOMETER SYSTEM

The properties of the traffic-serving surface of a pavement must be determined by measurement if the functional performance of the pavement is to be evaluated quantitatively. One of the devices which has been used for this purpose is the CHLOE Profilometer (Fig. 3). Figure 4 shows the element of the profilometer which senses the profile of the pavement. It consists of two solid disc wheels and related equipment mounted immediately in front of the pneumatic-tired wheels which support the rear of the profilometer boom.

Two other measuring devices are used in conjunction with the Profilometer for determining the properties of the surface of a pavement. These devices are the rut depth gauge (Fig. 5), which is used to measure the depth of wheel path ruts (*6*), and the Texturemeter (Fig. 7), which is used to measure the surface texture of a pavement (*2*).

The CHLOE Profilometer measurement system consists of a CHLOE Profilometer, including a special trailer to transport it (Fig. 6) (*3*), a towing vehicle, a rut depth gauge, and a texturemeter (see Fig. 2 for block diagram). In this manual it is assumed that the towing vehicle has been wired so that the vehicle generator is used to charge the Profilometer batteries and has been equipped with an AC-DC convertor to operate a small automatic calculator. It is also assumed that the towing vehicle is equipped with three trailer hitches, one at the center of the vehicle, one over the right wheelpath, and one over the left wheelpath.

The measurement system provides a method of obtaining the serviceability index of a section of pavement. This index, which was developed during the AASHO Road Test (*6*) is a quantitative measurement of the riding quality of a pavement surface. It is recommended that the following equations be used to calculate the serviceability index of pavements:

For rigid pavements

$$p = 5.41 - 1.80 \log (1 + \overline{SV}) - 0.09\sqrt{C + P} \qquad \text{(E-1)}$$

For flexible pavements

$$p = 4.85 - 1.91 \log (1 + \overline{SV}) - 0.01\sqrt{C + P} - 1.38\, \overline{RD}^2 + 0.81 \log (1 + T) \qquad \text{(E-2)}$$

in which:

- p = serviceability index;
- $\overline{SV}$ = slope variance determined with CHLOE Profilometer;
- $C + P$ = cracking plus patching;
- $\overline{RD}$ = average rut depth; and
- T = average texture determined with texturemeter.

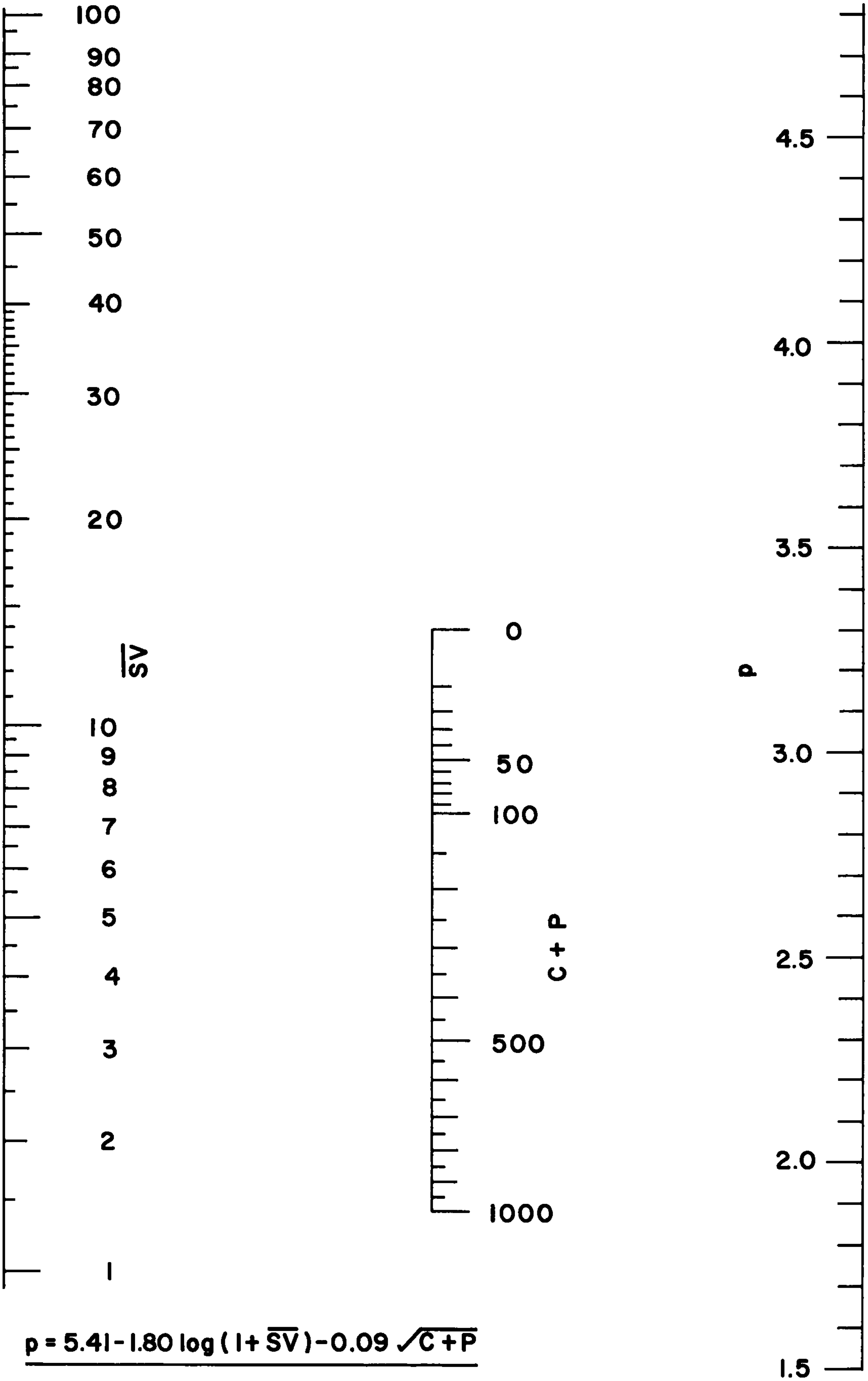
Eq. E-1, the AASHO equation for rigid pavements (*6*), is given in nomograph form in Figure E-1. Eq. 2, a modified form of the AASHO equation for flexible pavements (*2*) is given in nomograph form in Figure E-2.

Procedures for measuring slope variance, cracking, patching, rut depth, and texture are described in a subsequent section of this appendix. Other sections are concerned with checks to insure proper operation of the profilometer, and with methods of correcting the trouble when malfunctions are discovered.

PROCEDURES FOR CHECKING OPERATION OF PROFILOMETER

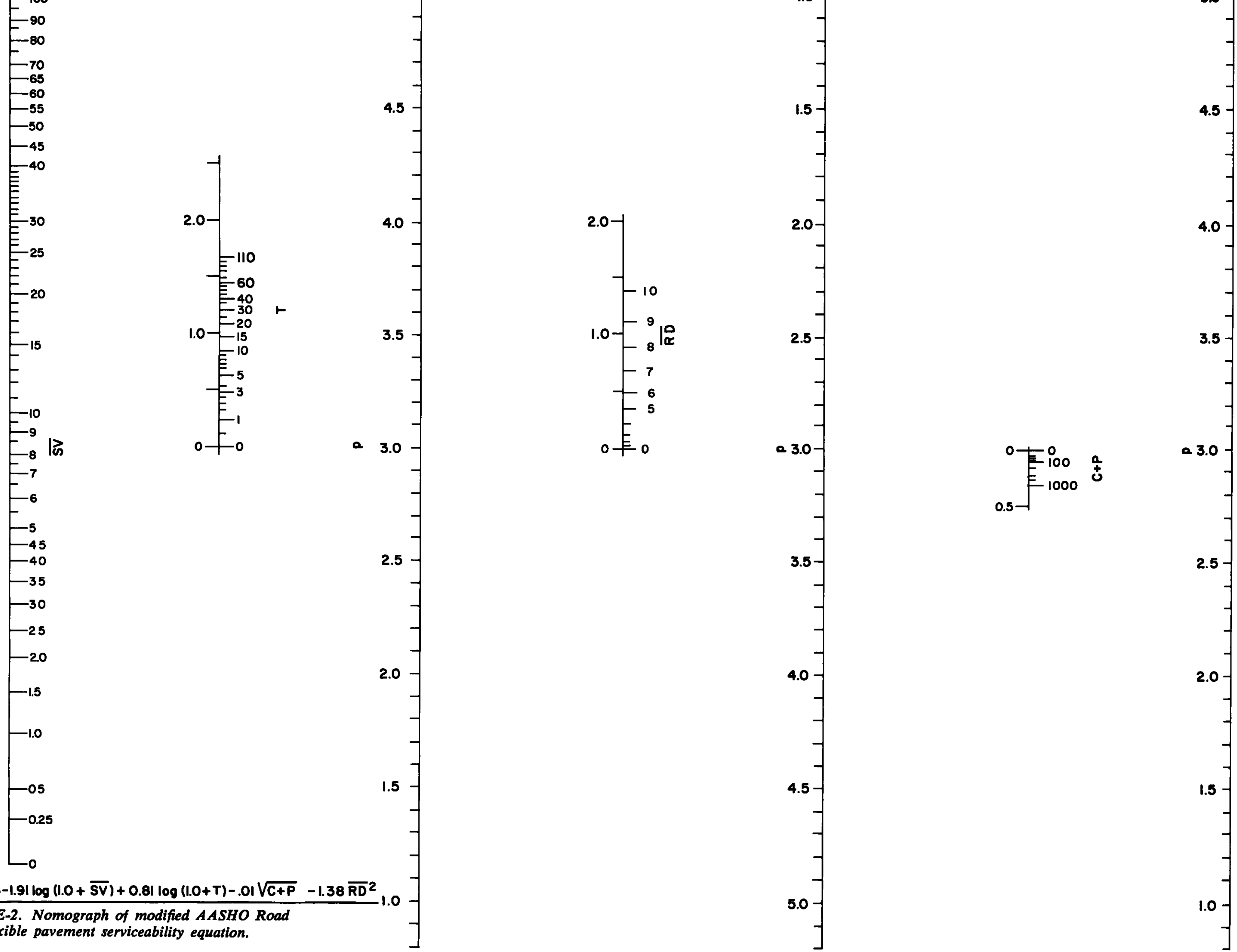
It is recommended that the following procedures be followed every day after arriving at the first section to be tested. They should also be followed at any other time when a malfunction is suspected. If a malfunction is observed, consult the later section on "Suggested Trouble-Shooting Points."

1. Attach Profilometer boom to vehicle.
2. Attach cables from boom to computer.
3. Switch battery plug from "charge" to "operate."
4. Turn on the computer and allow to warm up. Set both the power and count switches (Fig. E-3) to "on" positions. Reset counters to zero. Take the cover off the sensing component of the system (Fig. E-4). Raise the slope wheels above the surface of the pavement.
5. Disconnect electrical connector from wheel pulser and connect to manual pulser (Fig. E-4).
6. Manually place the roller on each successive switch contact in sequence and, using the manual pulse generator, generate one pulse on each placement of the roller (Figs. E-4 and E-5). The number of the switch and the square of the switch number will be displayed for each roller setting on the computer counters B and C (Fig. E-3). Reset counters before moving roller to next position. See Part 1 of Form P-1 (Fig. E-6).
7. Check ten (10) switch circuits at random by summing. Record readings after each pulse. Check the difference between switches; that is, compare differences between successive readings of counter B with the corresponding differences in counter C. See Part 2 of Form P-1 (Fig. E-6).
8. Check five (5) switch circuits. Place the roller on the switch contacts selected and, using the manual pulse generator, generate 100 pulses for each setting. Check the switch number sum and the sum of the squares of the switch numbers (Fig. E-3). See Part 3 of Form P-1 (Fig. E-6).
9. If the previous checks indicate equipment is functioning properly, proceed as indicated below. If any of the previous checks fails, consult section on "Suggested Trouble-Shooting Points."



$$p = 5.41 - 1.80 \log(1 + \overline{SV}) - 0.09\sqrt{C + P}$$

Figure E-1. Nomograph of AASHO Road Test rigid pavement serviceability equation.



$$P = 4.85 - 1.91 \log (1.0 + \overline{SV}) + 0.81 \log (1.0 + T) - .01 \sqrt{C+P} - 1.38\, \overline{RD}^2$$

Figure E-2. Nomograph of modified AASHO Road Test flexible pavement serviceability equation.

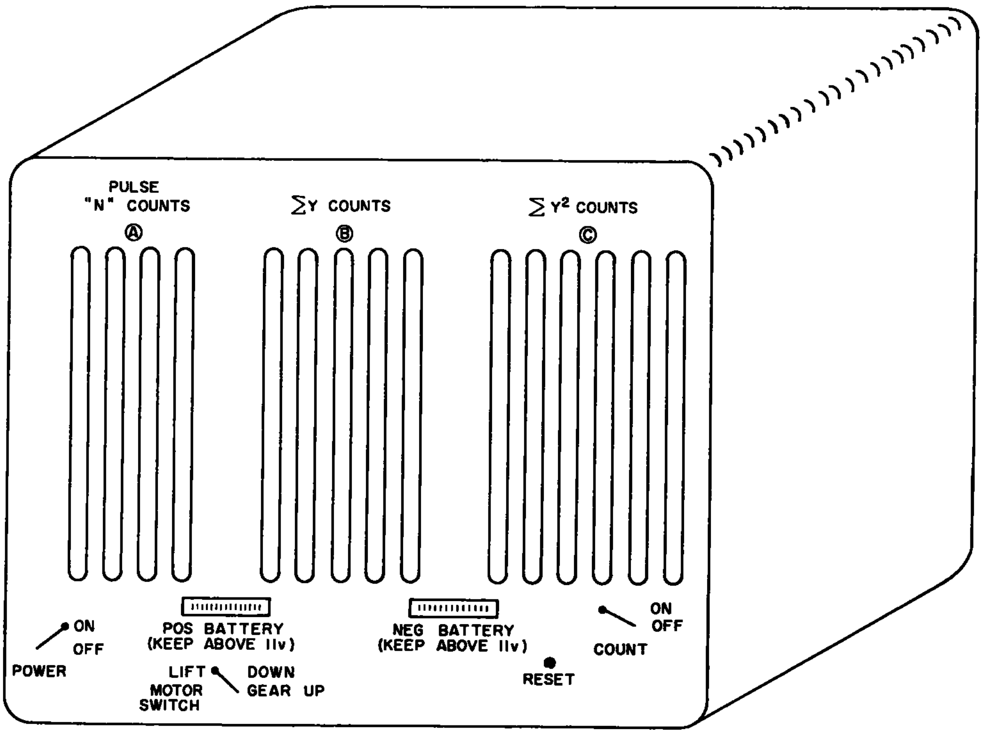


Figure E-3. Front of computer used with CHLOE Profilometer.

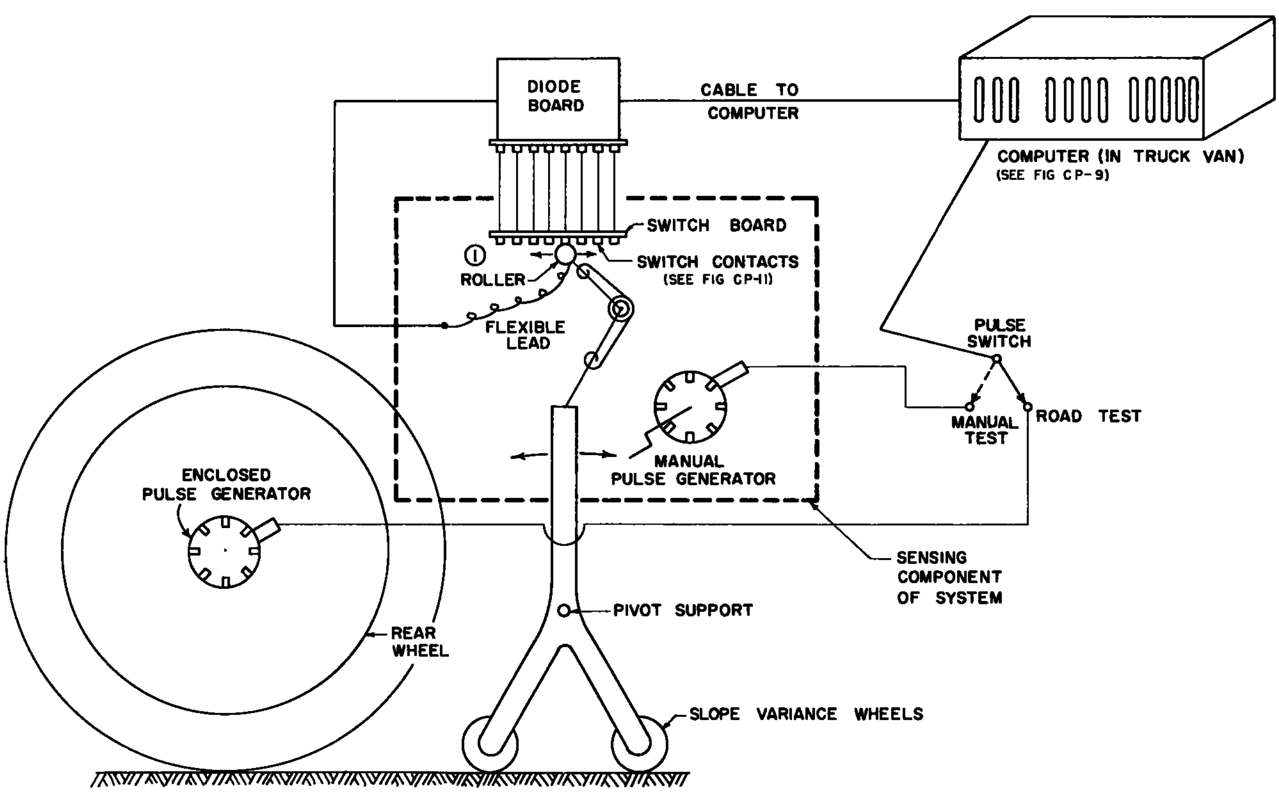


Figure E-4. Schematic diagram of CHLOE Profilometer.

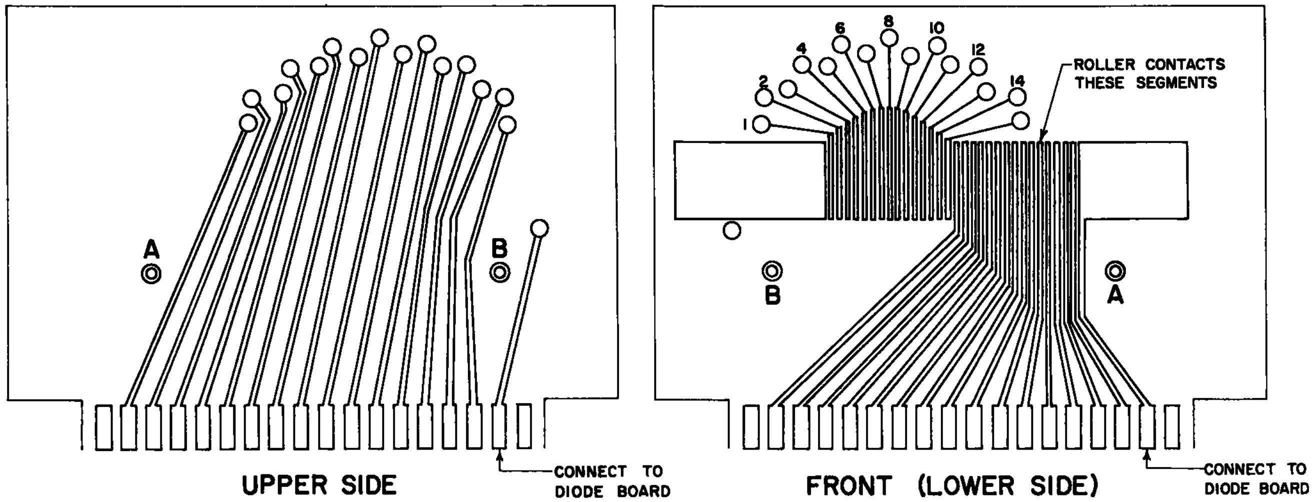


Figure E-5. Detail of switchboard.

10. Disconnect electrical cable from manual pulser and connect to wheel pulser.

11. Replace weatherproof cover on switching unit.

12. Pull onto pavement, lower profilometer slope wheels, and check profilometer boom for proper height.

PROCEDURES FOR MAKING MEASUREMENTS

Described in this section are the recommended procedures to be followed when testing sections that are laid out like the typical test section plan shown in Figure E-7. The procedures will necessarily have to be modified to accommodate other layouts.

Figures E-8 through E-11 are examples of field data forms. The particular data shown are those currently being used on the AASHO satellite study in Texas. They contain some coded information (for example, Cols. 69 through 80 on form P-5) (Fig. E-11), which, though appropriate to the Texas study, is not necessarily applicable to a study at the national level.

CHLOE Profilometer data are recorded on Form P-2 (Fig. E-8), which also serves as a guide for computing slope variance and serviceability index.

Cracking, patching, and rut depth measurements are recorded on Form P-3 (Fig. E-9); texturemeter readings are recorded on Form P-4 (Fig. E-10).

Form P-5 (Fig. E-11), which summarizes the data from several test sections, is usually filled out at the end of each day's operation. IBM cards (one for each test section) may be punched directly from this form.

Profilometer

In the procedure described in the following it is assumed that there are two operators—a driver and a recorder—in the towing vehicle during the test.

1. Turn on vehicle lights and AC-DC convertor.

2. Place proper identification of section, date, etc., on data sheet, Form P-2 (Fig. E-8).

3. Select inner or outer wheelpath for first test by flip of coin.

4. Connect Profilometer to vehicle.

5. Make computer, batteries, and Profilometer cable connections.

6. Turn on power switch and computer counter.

7. Position tow vehicle on pavement approximately 50 ft from start of test so that the slope wheels are in the selected wheelpath.

8. Lower the slope wheels to the pavement.

9. Drive toward "begin test" mark at a minimum speed of 2 mph but not more than 5 mph. Drive at the minimum speed on rough-textured pavements.

10. Hold reset button down until slope wheels cross "begin test" mark. Release button to start test.

11. Turn off counter after 600 ft of travel, then stop vehicle.

12. Record computer output on Form P-2 (Fig. E-8).

13. Raise slope wheels, and back up 50 ft.

14. Change profilometer to inner wheel path if outer path was tested initially, or vice-versa.

15. Lower slope wheels and repeat Steps 9 through 12. Calculate $\Sigma y/n$, $\Sigma y^2/n$, $(\Sigma y/n)^2$, and differences for initial data enroute. The profilometer has now traversed 1,200 ft.

16. After recording data, turn on counter, proceed.

17. Hold reset button down until slope wheels cross the 1,300-ft mark, then release to begin third test run. Make calculations for second run data while making the third 600-ft run. Calculate total difference $\times$ 4.23 and $\overline{SV}$ for first 1,200-ft subsection; enter on Form P-2 (Fig. E-8).

18. Turn off counter when recorder wheels cross end of third 600-ft run. Record data on second Form P-2 (Fig. E-8).

19. Stop truck and lift slope wheels. Back up 50 ft

FORM-P-1

DAILY CALIBRATION SHEET

CHLOE Profilometer

Date: 5-13-63 Section: TEST

1. CHECK OF 29 SWITCHES (Reset after each switch has been checked.)

y	y^2	√ if OK	y	y^2	√ if OK	y	y^2	√ if OK
1	1	✓	11	121	✓✓	21	441	✓✓
2	4	✓	12	144	✓	22	484	✓
3	9	✓	13	169	✓	23	529	✓
4	16	✓	14	196	✓	24	576	✓
5	25	✓	15	225	✓	25	625	✓
6	36	✓✓	16	256	✓✓	26	676	✓✓
7	49	✓	17	289	✓	27	729	✓
8	64	✓	18	324	✓	28	784	✓
9	81	✓	19	361	✓	29	841	✓
10	100	✓	20	400	✓			

2. CHECK OF SUMS (10 switches selected at random.)

n	Σy	Δy	Σy^2	Δy^2
1	29	29	841	841
2	56	27	1570	729
3	80	24	2146	576
4	101	21	2587	441
5	121	20	2987	400
6	138	17	3176	289
7	151	13	3345	169
8	161	10	3445	100
9	167	6	3481	36
10	169	2	3485	4

3. CHECK OF RAPID PULSING

Select a switch at random. Pulse rapidly 100 times. If n, Σy and Σy^2 are correctly indicated put additional check in Table 1 above.

Repeat for a total of 5 switches.

Figure E-6. Daily calibration sheet for CHLOE Profilometer.

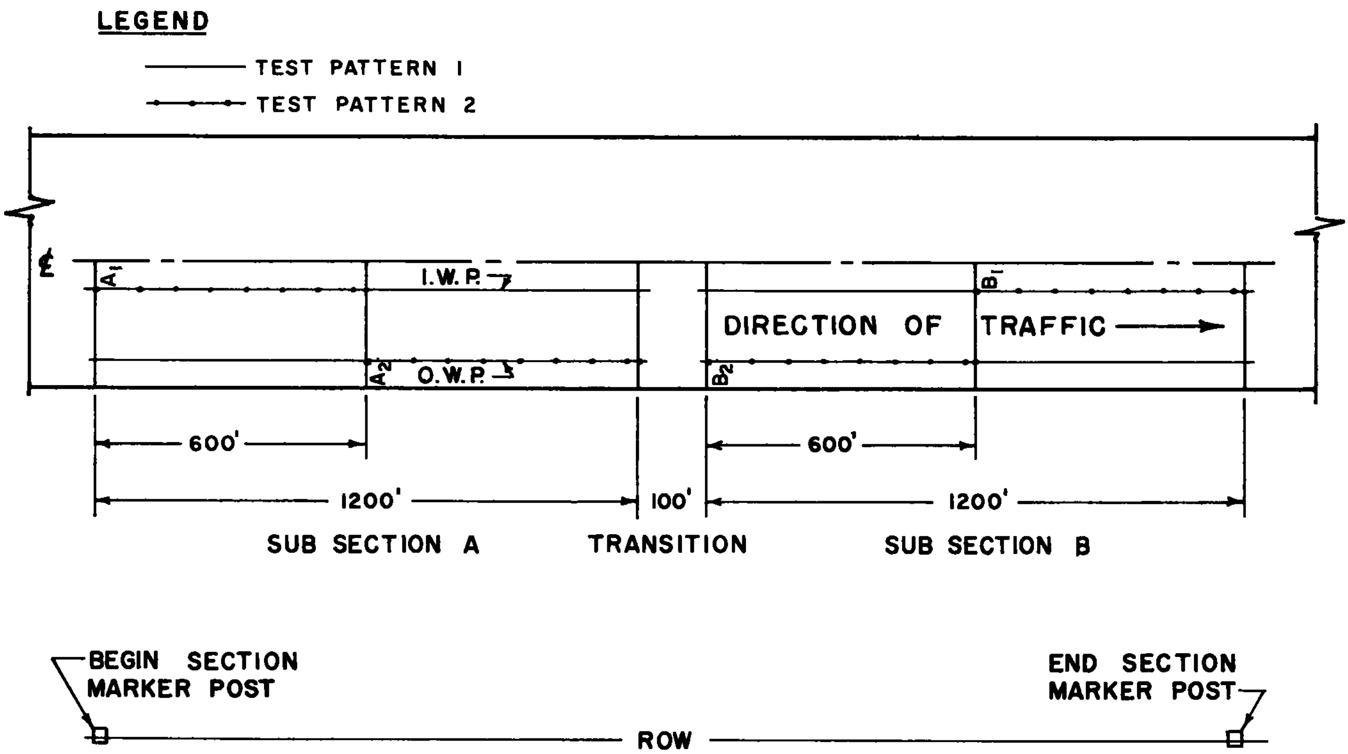


Figure E-7. Typical test section plan.

behind start of fourth run and move profilometer into opposite wheelpath.

20. Lower slope wheels, turn on counter and hold down reset button until slope wheels cross the "begin test" mark for last 600-ft run.

21. Calculate values listed in Step 15 for third test run while conducting fourth run.

22. Turn off counter when slope wheels cross end of section. Record data on second Form P-2 (Fig. E-8).

23. Lift slope wheels, turn off power to computer, drive onto shoulder of roadway, disconnect cables and Profilometer. Load Profilometer onto trailer for transporting to next test section.

24. Calculate data taken on fourth test run and total difference $\times$ 4.23 and $\overline{SV}$ for last 1,200-ft subsection.

25. If the profilometer is not loaded on the trailer, it may be towed short distances on its own pneumatic-tired wheels at speeds not over 20 mph provided the *slope wheels are in the up position.*

Rut-Depth Gauge

The rut depth gauge, a hand-operated instrument, furnishes a measurement of the permanent deformation in the wheelpaths of a flexible pavement.

Five or six measurements are made in each wheelpath in each subsection. These data are entered on Form P-3 (Fig. E-9) along with estimates of cracking and patching.

Cracking and Patching

Measurements of cracking and patching are made in accordance with the rules used at the AASHO Road Test and described in Ref. (*6*, p. 23) for flexible, and (*6*, p. 142) for rigid pavements. These rules are repeated below.

In the case of rigid pavements, cracking, *C* (Eq. E-1), is defined as the total linear feet of class 3 and class 4 cracks per 1,000 sq ft of pavement area. The length of a crack is taken as the length of its projection parallel or perpendicular to the pavement centerline, whichever is greater. A class 3 crack is defined as a crack opened or spalled at the surface to a width of ¼ in. or more over a distance equal to at least one-half the crack length, except that any portion of the crack opened less than ½ in. at the surface for a distance of 3 ft or more is classified separately. A class 4 crack is defined as any crack which has been sealed. Patching, *P*, is expressed in square feet per 1,000 sq ft of pavement surfacing.

For flexible pavements, cracking, *C* (Eq. 2), is defined as the area, in square feet per 1,000 sq ft of pavement surface, exhibiting class 2 or class 3 cracking. Class 2 cracking is defined as that which has progressed to the stage where cracks have connected together to form a grid-type pattern. Class 3 cracking is that in which the bituminous surfacing segments have become loose. Patching, *P*, is the repair of the pavement surface by skin patching or deep patching expressed in square feet per 1,000 sq ft of pavement surfacing.

FORM-P-2

PROFILOMETER DATA

Field Form 1 for Serviceability Index
Profilometer Survey

Flexible [✓] $P = 4.85 - 1.91 \log (1 + \overline{SV}) - .01 \sqrt{C + P} - 1.38\ \overline{RD}^2 + 0.81 \log (1 + T)$

Rigid [ ] $P = 5.41 - 1.80 \log (1 + \overline{SV}) - .09 \sqrt{C + P}$

Section No. 866-2-1 A * District No. 6 *

County CRANE Highway F.M. 1053

Date 5-13-63 * Crew Chief ARMSTRONG, J.C.

Arrival Time 11:30 A.M. * Pavement Texture SMOOTH

Beginning Station 0 Ending Station 12

Direction of Travel NORTH

Profilometer Readings

W.P. & Sequence	Inner (2)	Outer (1)	Observed	Calculated
n	1168	1176	✓	
y	20635	16326	✓	
y^2	365551	227427	✓	
y/n	17.6670	13.8827		✓
$\Sigma y^2/n$	312.9718	193.3903		✓
$(\Sigma y/n)^2$	312.1229	192.7294		✓
Difference	0.8489	0.6609		✓

Σ Diff. x 4.23 6.3865

Correction -2.5000

$\overline{SV}$ 3.8865 *

C + P 10.4 *

$\overline{RD}$ 0.1 *

Texture 1.2 *

Serv. Index = P = 3.70

Equation Solved by Nomograph
(Computation checked in computer program)

*Entered on Form-P-5

Figure E-8. Profilometer survey data sheet, field form 1 for serviceability index.

FORM-P-3

PROFILOMETER DATA

Field Form 2 for Serviceability Index

Condition Survey

(Staple completed form to Form 1)

Section No. 866-2-1A Date 5-13-63

Section Length (ft.) 1200 Joint spacing (Rigid) FLEX.

Lane Width (ft.) 12 Surveyor C.E.S.

CRACKING (C)	PATCHING (P)	RUT DEPTH IWP	RUT DEPTH OWP
Ft. (Rigid) Sq. Ft. (Flex)	Sq. Ft.	(0.1 inches (Flex))	
55	95	1	
			1
		1	
			2
		1	
			1
		2	
			1
		1	
			1
		1	
ΣC = 55 sq.ft.	ΣP = 95 sq.ft.	ΣRD = 13 ins.	

I. ΣC + ΣP = 150 sq.ft. IV. No. RD Meas. 11

II. Sect. Area (Sq. ft/1000) 14.4 V. $\overline{RD} = \frac{(0.1)\Sigma RD}{IV} = \frac{0.1(13)}{11} = 0.12$ in.**

III. C + P = I/II = $\frac{150}{14.4} = 10.4$ sqft/1000 sqft**

**Entered on FORM-P-2

Figure E-9. Condition survey data sheet, field form 2 for serviceability index.

FORM-P-4

PROFILOMETER DATA

FIELD RECORD OF PAVEMENT TEXTURE MEASUREMENTS

District 6 County CRANE Highway F.M. 1053

Section 866-2-1 Date 5-13-63 Meas. by E. H.

Time 11:30 A.M.

Location*	Sampler	FIRST SUBSECTION (A) Outer	FIRST SUBSECTION (A) Inner	SECOND SUBSECTION (B) Outer	SECOND SUBSECTION (B) Inner
1	1	0	0	0	0
	2	1	0	2	1
2	3	4	1	2	3
	4	1	5	1	1
3	5	1	0	1	2
	6	0	2	1	1
4	7	2	0	0	2
	8	1	2	2	1
5	9	1	1	1	2
	10	2	0	0	3
	Average:	1.3	1.1	1.0	1.6
	Outer Average:		+1.3		+1.0
	Sums:		2.4		2.6
	Subsection Averages:		1.2 **		1.3 **

*Space sample locations at about 200 feet.
**Entered on FORM-P-2

Figure E-10. Field record of pavement texture measurements.

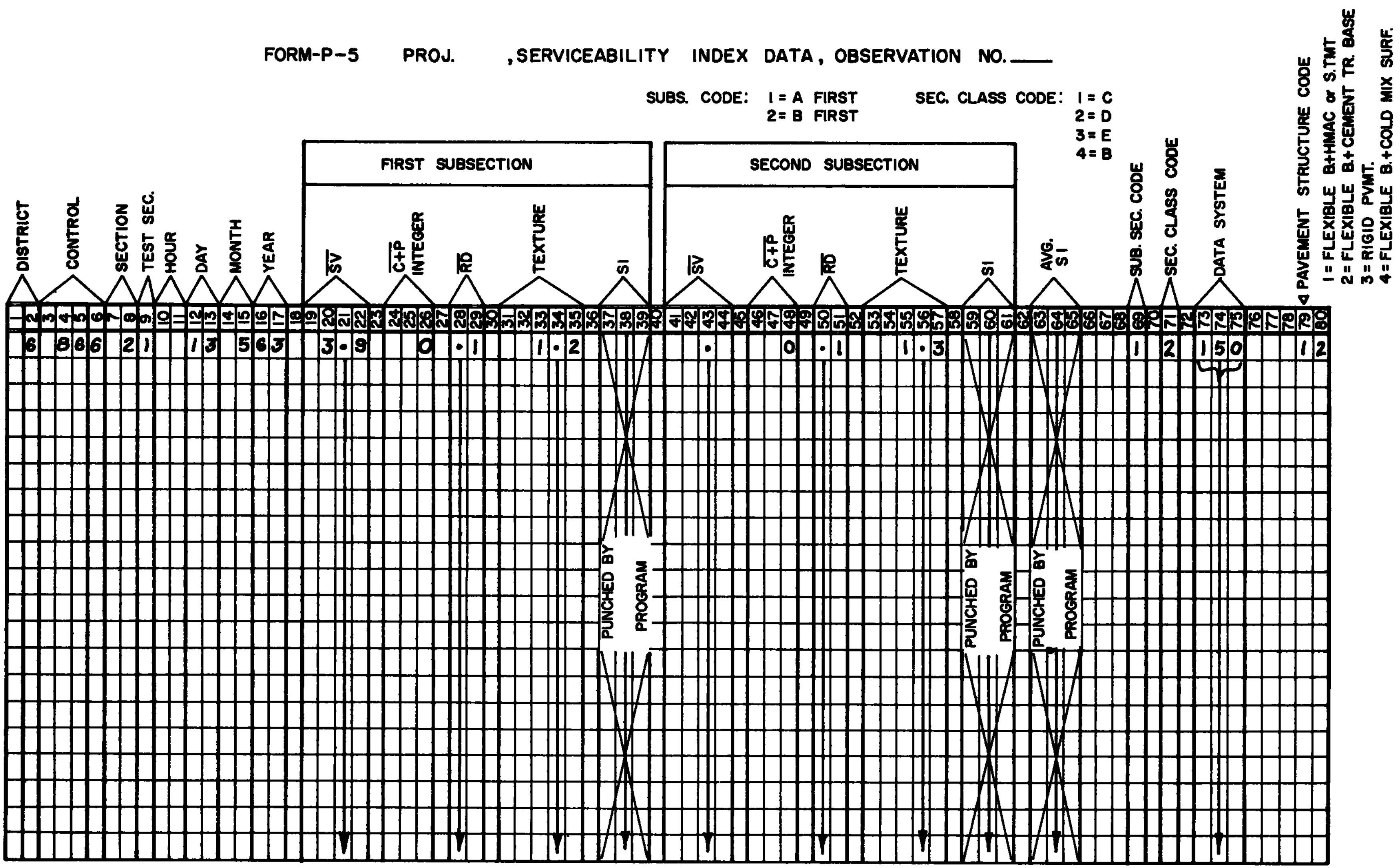


Figure E-11. Serviceability index data sheet used with CHLOE Profilometer measurements.

Cracking and patching data are entered on Form P-3 (Fig. E-9).

Texturemeter

The Texturemeter (*2*), a hand-operated instrument, furnishes a measure of unevenness of the surface of flexible pavements due to aggregate projecting from the surfacing material.

Prior to use on each subsection, this instrument should be zeroed by pressing it downward on a flat metallic surface. Ten Texturemeter readings are then taken at approximately evenly spaced points in each wheelpath in the subsection. To avoid bias, the exact location at which a reading is taken should be a random choice. Readings are recorded on Form P-4 (Fig. E-10).

SUGGESTED TROUBLE-SHOOTING POINTS

The following list of possible malfunctions of the CHLOE Profilometer, and suggested methods for locating the source of the trouble, may be of assistance if any one of the checks listed under "Procedures for Making Measurements" fails, or if erratic behavior is observed during the measurement of slope variance on a test section.

It is assumed that a member of the crew has some knowledge of electronics and has a set of spare circuit boards for the computer. A recommended list of spare circuit boards is also given.

SPARE CIRCUIT BOARDS FOR COMPUTER [a]

ITEM NO.	NAME OF BOARD	BOARD NO.	LOCATION (SLOT NO.)
1	Distance Pulse Marker	B213	1
2	Switching Circuit	B212	2 through 4
3	D. C. and Gate	B214	5 and 6
4	R. S. Flip Flop Assembly-Storage	B215A	7 through 10
5	Emitter Follower	B217	11 through 13 and 20
6	Clock Generator	B205	14
7	R. S. Flip Flop Assembly-Scanner	B215	15 through 17
8	Pulse or Circuit	B219	18
9	Pulse or Gate	B218	19
10	D. C. and Gate	B216	22 through 32
11	Delay Circuit	B208	33
12	Decade Pulse or Gate	B206	34
13	Decade Unit	B222	35 through 49
14	Power Supply	Bd25	50

[a] This list applies to the early model CHLOE Profilometer used on the Texas satellite project. The list for later models probably differs somewhat.

MALFUNCTION	SUGGESTED METHOD OF LOCATING
A. Computer does not "light up."	1. Check connections (power). 2. Check fuses. 3. Replace power supply circuit board in computer with spare. 4. Trouble-shoot computer. Note: Consult competent electronics specialist.
B. No count on computer.	1. Check fuses in computer. 2. Check to assure roller is in contact with switch board. 3. Check cable connections. 4. Check for broken wires in the switching unit. 5. Check for operation of diode lights in pulser units (includes pickup cells). 6. Check for broken pulser wire in cable. 7. Check for excessive moisture in pulser unit.
C. Non-uniform counts or apparent erratic results.	1. Check battery condition. 2. Check condition of roller and switch board. 3. Check cable connections. 4. Check for broken or shorted wires in the switching unit and pulser unit. 5. Check for broken or shorted wires in cables. 6. Check height of boom. 7. Check for low temperature. 8. Check for water or excessive moisture in wheel pulser unit. 9. Replace circuit boards in computer with spares. 10. Trouble-shoot computer. Note: Consult competent electronics specialist.
D. Motor will not raise or lower boom.	1. Check battery voltage. 2. Check fuses. 3. Check cables. 4. Check for binding parts. 5. Check for faulty motor.
E. Excessive vibration of Profilometer.	1. Check height of boom. 2. Check speed of vehicle. 3. Check for bent or binding parts.

APPENDIX F

OPERATIONS MANUAL FOR DYNAFLECT SYSTEM

The Lane-Wells Dynaflect is a device that induces and measures the deflection of a pavement surface. Mounted on a small, two-wheel trailer (Fig. 11), the Dynaflect can be towed at normal highway speeds by a passenger automobile and stopped briefly at a test location to make deflection measurements.

The load which produces the deflection is generated by a pair of counter-rotating weights and is applied to the pavement through two steel wheels (Fig. 11). Deflections are sensed by geophones (Fig. 19) lowered to the pavement by an electrically operated pulley system. The geophones actuate a meter (shown to the left in Fig. 12) on which the deflection sensed by each geophone can be read. A calibrator unit (Fig. F-1) completes the essential parts of the system. Operated by remote control from the front seat of the towing vehicle, the Dynaflect requires only one operator.

A comprehensive manual containing detailed descriptions and drawings of parts, as well as operating instructions and trouble shooting procedures, is available from the manufacturer on request.* This report, therefore, is restricted to the major operating procedures and to the methods of data handling believed appropriate for the national satellite program.

CALIBRATION PROCEDURE

This section contains the general procedures for calibration that should be followed upon arriving at each test section. A detailed description of these procedures, as well as the procedures to be followed when a malfunction is observed, are contained in the manufacturer's operations manual.*

1. Connect the calibrator (Fig. F-1) to control unit (Fig. 12). Connect five geophones to their mating connectors and put geophones 1 through 4 in the calibrator.
2. Turn on power switch and allow control unit to warm up. Place frequency toggle switch in calibrate position, sensor toggle switch in down position, and deflection multiplier in "cal" position. Adjust calibrator control frequency to 8 cps.
3. Place sensor selector switch to position 1 and adjust sensor 1 trim knob so that the deflection meter reads at the "cal" position. Lock sensor 1 trim knob.
4. Place sensor selector switch to position 2. Adjust and lock sensor 2 trim knob. Similarly adjust sensors 3 and 4. Turn off sensor selector switch.
5. Replace one of geophones in the calibrator with sensor 5 and adjust sensor 5 using the same procedure as used for sensors 1 through 4.
6. Recheck frequency meter and readings for the geophones in calibrator. Place sensor toggle switch in up position.
7. Disconnect and stow the calibrator.

PROCEDURES FOR MAKING MEASUREMENTS

Described in this section are the recommended procedures to be followed when using five sensors to define the deflec-

* "Operators Manual for the Dynaflect." Lane-Wells Co., Box 3386, Houston, Tex. 77001.

* Ibid.

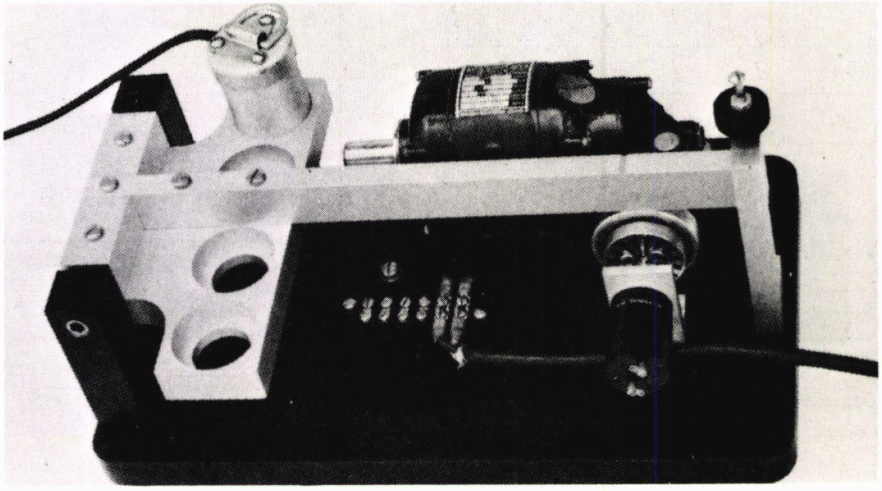


Figure F-1. Unit for calibrating geophones.

tion basin produced by the Dynaflect. It is recommended that the geophones be placed on the geophone carriage so that they will be placed on the pavement surface as shown in Figure 19. It is also recommended that within each test section, measurements be made at 15 equally spaced test points in the outer wheelpath.

Forms LW-1 (Fig. F-2) and LW-2 (Fig. F-3) are examples of field data forms. The particular data forms shown are those currently being used on the AASHO satellite study in Texas. They contain some information which, though appropriate to the Texas project, are not necessarily applicable to a study at the national level. Figure F-2 is used to record Dynaflect data observed on one test section. Figure F-3, which summarizes the data from several test sections, is usually filled out at the end of each day's operation. IBM cards (five for each test section) may be punched directly from Form LW-2.

A detailed description of the Dynaflect operating proce-

Form-LW-1

DYNAFLECT DATA SHEET

District 12 Section 110-3-1 Hour 2:00 Day 14 Month MAR Year 66

Temperature 86 County MONTGOMERY Subs. Code 2 (1=A First; 2=B First)

Highway US 75 Direction of Travel N Measured by RP

	Sensor No. 1			Sensor No. 2			Sensor No. 3			Sensor No. 4			Sensor No. 5		
Station	Read	Mult	Def	Read	Mult	Def	Read	Mult	Def	Read	Mult	Def	Read	Mult	Def
1	5.4	.1	.54	2.9	.1	.29	4.7	.03	.14	2.8	.03	.08	1.7	.03	.05
2	7.3	.1	.73	4.1	.1	.41	1.8	.1	.18	3.6	.03	.11	2.4	.03	.07
3	5.0	.1	.50	2.7	.1	.27	4.8	.03	.14	3.2	.03	.10	2.3	.03	.07
4	5.9	.1	.59	3.1	.1	.31	4.4	.03	.13	2.6	.03	.08	4.6	.01	.05
5	6.7	.1	.67	3.8	.1	.38	2.2	.1	.22	5.0	.03	.15	3.3	.03	.10
6	7.6	.1	.76	4.4	.1	.44	2.3	.1	.23	4.5	.03	.14	2.7	.03	.08
7	4.2	.1	.42	2.1	.1	.21	3.2	.03	.10	2.1	.03	.06	3.9	.01	.04
8*	4.5	.1	.45	6.6	.03	.20	3.0	.03	.09	2.1	.03	.06	1.4	.03	.04
9	4.7	.1	.47	2.2	.1	.22	3.0	.03	.09	2.2	.03	.07	4.2	.01	.04
10	5.7	.1	.57	3.1	.1	.31	5.0	.03	.15	2.9	.03	.09	1.7	.03	.05
11	4.8	.1	.48	2.7	.1	.27	4.3	.03	.13	2.8	.03	.08	1.7	.03	.05
12	5.2	.1	.52	2.9	.1	.29	4.7	.03	.14	3.0	.03	.09	1.9	.03	.06
13	3.8	.1	.38	2.5	.1	.25	5.3	.03	.16	3.7	.03	.11	2.6	.03	.08
14	4.7	.1	.47	2.8	.1	.28	1.8	.1	.18	4.2	.03	.13	3.0	.03	.09
15	5.0	.1	.50	2.9	.1	.29	5.7	.03	.17	4.0	.03	.12	2.5	.03	.08

*Station 8 to be located at point where Shell Vibrator measurements were made (usually in the transition between sub-sections).

REMARKS:
NONE

Figure F-2. Dynaflect field data record.

Form-LW-2

PRODUCTION DYNAFLECT DATA

Geo. Code: 1 = 0, 2 = 1', 3 = 2', 4 = 3', 5 = 4'

Subs. Code: 1 = A first, 2 = B first

Sec. Class Code: 1 = C, 2 = D, 3 = E

District (1–2)	Control (3–6)	Section (7–8)	Test Sec. (9)	Hour (10–11)	Day (12–13)	Month (14–15)	Year (16–17)	Temp. (18–20)	Deflection in Milli-inches at Station Indicated: 1 (21–23)	2 (24–26)	3 (27–29)	4 (30–32)	5 (33–35)	6 (36–38)	7 (39–41)	8 (42–44)	9 (45–47)	10 (48–50)	11 (51–53)	12 (54–56)	13 (57–59)	14 (60–62)	15 (63–65)	(66)	Geo. Code (67)	(68)	Sub. Sec. Code (69)	(70)	Sec. Class Code (71)	(72)	Data System (73–75)	(76)	(77)	(78)	(79)	(80)
12	110	3	1	14	14	3	66	86	054	073	050	059	067	076	042	045	047	057	048	052	038	047	050		1		2		3		310					
←		REPEAT						→	029	041	027	031	038	044	021	020	022	031	027	029	025	028	029		2		2		3		310					
←		REPEAT						→	014	018	014	013	022	023	010	009	009	015	019	014	016	018	017		3		2		3		310					
←		REPEAT						→	008	011	010	008	015	014	006	006	007	009	008	009	011	013	012		4		2		3		310					
←		REPEAT						→	005	007	007	005	010	008	004	004	004	005	005	006	008	009	008		5		2		3		310					
		REPEAT																							1						310					
←		REPEAT						→																	2						310					
←		REPEAT						→																	3						310					
←		REPEAT						→																	4						310					
←		REPEAT						→																	5						310					
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←		REPEAT						→																	3						310					
←		REPEAT						→																	4						310					
←		REPEAT						→																	5						310					

Figure F-3. Production Dynaflect data sheet.

dure is contained in the manufacturer's operations manual.* Only the major operational steps performed on a test section are listed in the following.

1. Place identification of section, date, etc., on Dynaflect data sheet (Fig. F-2).
2. Screw triangular bases on geophones and connect the geophones to the geophone carriage.
3. Turn on vehicle warning lights and pull onto pavement.
4. Place frequency toggle switch to operate position and place force toggle switch to down position.
5. Drive to first test point.
6. Place sensor switch in down position and adjust frequency to 8 cps.
7. Place sensor selector switch in position 1 and adjust multiplier switch for maximum reading. Record deflection meter reading and multiplier switch setting on data sheet. Repeat procedure for sensors 2 through 5.
8. Place sensor switch in up position.
9. Repeat Steps 5 through 8 for test points 2 through 15.
10. Place force switch in up position.
11. Drive off pavement, turn off power switch, and turn off warning lights. Disconnect and stow geophones.

* Ibid.

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Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

THE NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

THE NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U. S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

THE DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

THE HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering, is a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.