NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT 22

FACTORS INFLUENCING FLEXIBLE PAVEMENT PERFORMANCE

HIGHWAY RESEARCH BOARD NATIONAL ACADEMY OF SCIENCES - NATIONAL RESEARCH COUNCIL

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FACTORS INFLUENCING FLEXIBLE PAVEMENT PERFORMANCE

ROBERT L. KONDNER AND RAYMOND J. KRIZEK
THE TECHNOLOGICAL INSTITUTE, NORTHWESTERN UNIVERSITY
EVANSTON, ILLINOIS

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

SUBJECT CLASSIFICATIONS:
PAVEMENT DESIGN
PAVEMENT PERFORMANCE

HIGHWAY RESEARCH BOARD OF THE DIVISION OF ENGINEERING
NATIONAL ACADEMY OF SCIENCES - NATIONAL RESEARCH COUNCIL 1966

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 Highway Planning and Research 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 Commerce.

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 1-3(2) FY '63
NAS-NRC Publication 1221
Library of Congress Catalog Card Number: 66-60095

FIGURES

ACKNOWLEDGMENTS

Those who have worked on this project include Robert L. Kondner, Associate Professor of Civil Engineering and Raymond J. Krizek, Assistant Professor of Civil Engineering, The Technological Institute, Northwestern University, who served, respectively, as Project Director and Project Engineer.

The assistance of Nausherwan Hasan, Eiichi Yamamoto, and Joseph Zelasko in various phases of the research is gratefully acknowledged.

The Institute furnished facilities and support conducive to the effective prosecution of the work.

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ACKNOWLEDGMENTS

This report closes the initial phase of a project involving the continuing cooperation and contributions of many organizations and individuals. For this phase, some of the following provided both data—often at no cost to the project—and valuable commentary. Others reviewed preliminary draft material. In most instances, not only did the participants named contribute freely, but so also did members of their staffs. Much appreciation is expressed to all.

Special thanks are extended to David K. Witheford, Transportation Planning Consultant, West Haven, Conn., who not only participated in the project during 1964 and 1965, but who also is chiefly responsible for the chapter and appended data on shopping centers.

Basic origin-destination and supporting transportation study data were provided by H. H. Huckeba and his successor Emory C. Parrish, State Planning Engineers, Georgia State Highway Department, for the Atlanta Area Transportation Study; Roger L. Creighton, Director, Subdivision of Transportation Planning and Programming, and John R. Hamburg, Director, Bureau of Planning, New York Department of Public Works, for the Niagara Frontier Transportation Study; E. Wilson Campbell, Director, Chicago Area Transportation Study; T. C. Reseigh, Planning and Research Engineer, Colorado Department of Highways, for the Denver Metropolitan Area Transportation Study; Edward T. Telford, Assistant State Highway Engineer, California Division of Highways, for the Los Angeles Regional Transportation Study; Floyd B. Bowen, Chairman, Florida State Road Department, and Mel Conner, President, Mel Conner & Associates, for the Miami Urban Area Transportation Study; Johan Nygaard, Planning Research Engineer, Minnesota Department of Highways, for the Twin Cities Area Transportation Study; David D. Longmaid, Executive Director, Penn-Jersey Transportation Study; Robert Kochanowski, Director, Pittsburgh Area Transportation Study; Richard J. Bouchard, Director, Rhode Island Statewide Comprehensive Transportation and Land Use Planning Program; Gerald Matthews, Transportation and Research Engineer, Salt Lake Area Transportation Study; Martin J. Bouman, Transportation and Traffic Engineer, City of San Diego, and C. G. Beer, Urban Planner, California Division of Highways, for the San Diego Traffic Survey; John K. Mladinov, Director, Puget Sound Regional Transportation Study; G. E. Marple, Chief, Urban Planning Division, U. S. Bureau of Public Roads, for the 1955 Washington Metropolitan Area Transportation Study; and George V. Wickstrom, Program Director, New Castle County Program, and Ernest A. Davidson, Director of Operations, Delaware State Highway Department, for the Wilmington study data.

Airport site and statistical data were provided by Grady H. Ridgeway, Jr., Manager, Atlanta Municipal Airport; Michael J. Berry, Manager, Chicago Midway Airport; Alan C. Stewart, Director, Miami International Airport; L. A. Johnson, Director, Minneapolis-St. Paul International Airport; Eugene J. Kuhn, Airport Development Assistant, Philadelphia International Airport; J. W. Carlson, Director, Greater Pittsburgh Airport; Albert R. Tavani, Administrator of Aeronautics, Theodore Francis Green State Airport; Joe Bergin, Manager, Salt Lake Municipal Airport No. 1; Dale H. Fearn, Manager, San Diego Municipal Airport; Arthur H. Yoshioka, Director of Planning and Research, Port of Seattle; and E. B. Franklin, Manager, Washington National Airport.

Thanks also are owing to some thirty plant managers, and to many shopping center managers or management agencies, who furnished information about their respective plants or shopping centers.

Helpful information pertaining to the particular cities in the cross section was also given by the directors of various city and regional planning agencies, directors of city engineering departments, chambers of commerce, newspapers, and other institutions.

Others who assisted in the project by providing suggestions and ideas, or by reviewing preliminary draft materials, were John W. Barr, G. William Blake, Harlan H. Blindauer, F. C. Burton, Donald D. Carroll, Gary R. Cowan, Charles C. Crevo, A. Keith Gilbert, Gayle L. Harder, Wolfgang S. Homburger, John J. Howe, Richard E. Hubbell, Matthew J. Huber, Robert Irwin, H. G. Kuitu, Louis Kwasniewski, Michael Lash, R. W. Pulling, Clyde E. Pyers, Ronald Sarras, and Deane M. Wenger.

FOREWORD

By Staff

Highway Research Board

Research engineers concerned with flexible pavement design will be those having most interest in this report. It contains the results of a study having the objective of establishing significant trends between flexible pavement response and various factors such as axle load, number of load applications, and thickness of pavement components. Performance data from the AASHO Road Test and other similar experiments have been examined, and observed behavioral trends have been expressed mathematically. The possibility of using this information in incorporating performance, as expressed by the present serviceability index (PSI), into pavement design procedures is discussed and illustrated by means of hypothetical examples.

The AASHO Road Test and previous similar experiments have provided a substantial fund of information descriptive of the fundamental factors concerned with the subject of pavement design and performance. The degree of influence of the factors commonly assumed to affect performance has not been suitably evaluated, however, to allow translation of performance test results from one area to another.

While other research is concerned with organizing regions into like groupings of sufficient size to permit the applications of the principles of meteorology, pedology, geology, and geomorphology to the identification of significant factors influencing pavement performance, the study at Northwestern University was concerned with factors "in the small" which influence flexible pavement performance. In pursuing this aspect, the research has dealt specifically with the factors of axle load, number of load applications, and the thickness of certain pavement components. On the premise that functional pavement design should recognize the relationship between these factors and performance, data from 134 AASHO Road Test flexible pavement sections were examined for behavioral trends. This has resulted in a mathematical expression on which it is theorized that performance, expressed in terms of PSI, can be incorporated into design procedures. This expression recognizes the factors of long-term and cyclic deterioration rates, and correlations have been made between PSI and such AASHO Road Test data as slope variance, rut depth, and roughness, and between the factors of interest and Benkelman beam deflection. The study further includes the use of nondimensional techniques in correlating the factors of interest with load-deflection behavior of flexible pavements during plate-bearing tests. Mathematical expressions of the response have been obtained, and correlations between actual and theoretical results are graphically illustrated in the report.

This is a final report in the general subject area of translating AASHO Road Test results to local conditions, and conclusions have been drawn concerning the existence of trends between flexible pavement performance and the various factors of interest;

the value of these trends in possibly contributing to improved design procedures, as well as in designing satellite test programs for the extrapolation of AASHO Road Test results to local conditions; and the limitations of these trends in their present stage of development. Throughout the report proper credence is given to the various conditions by which this study has been limited, and caution is urged against the wholesale application of the results to other areas. The developed hypotheses deserve further investigation to determine their ultimate applicability in terms of guidance for the design of various satellite test programs.

FACTORS INFLUENCING FLEXIBLE PAVEMENT PERFORMANCE

SUMMARY

Flexible pavement performance, as expressed by the present serviceability index (PSI), is related to local factors, such as surface course thickness, a, base course thickness, b, subbase course thickness, c, vehicle axle load, P, and number of load applications, N, for the AASHO Road Test results. It is given as

$$\mathrm{PSI}(N) = (\mathrm{PSI})_0 - C_1\, 10^{(C_2P-C_3)}\, 10^{-(C_4+C_5P)D}\, N - \sum_{i=1}^{k} H(N_i)\, C_6\, 10^{(C_7P-C_8)}\, 10^{-(C_9+C_{10}P)D}$$

in which $\mathrm{PSI}(N)$ is the present serviceability index of a given section after N load applications, $(\mathrm{PSI})_0$ is the PSI at $N = 0$, $H(N_i)$ is the Heaviside unit step function, k is the number of spring thaws which occur during the N load applications, parameters C_1 through C_{10} are numerical coefficients, and D is the thickness index which is written as $D = 0.44\,a + 0.14\,b + 0.11\,c$. Correlation plots with the AASHO Road Test data are presented for this equation as well as the component equations used in its development. The possibility of incorporating PSI into pavement design procedures, using these equations in association with local satellite test programs, is discussed and illustrated with several hypothetical examples.

The above equation for PSI as a function of number of years in service gives highway performance directly in terms of such local design factors as the thicknesses of the components of the flexible pavement system and axle load, whereas the equation previously developed by the AASHO Road Test staff gives PSI only in terms of physical measurements (such as slope variance, rut depth, and cracking plus patching) expressing a state of structural deterioration after the highway has been built and in use. Thus, the present developments present the idea of determining PSI before construction from the design variables. This is an important step forward because it advances the concept of specifying performance as a design criterion and selecting a flexible pavement system design to meet the performance criterion. However, the numerical coefficients should be specified in accordance with local climatic and soil conditions.

Flexible pavement performance in terms of PSI is correlated with various physical measurements obtained from the AASHO Road Test. These include slope variance, SV, rut depth, RD, and roughness, R.

Correlations of local factors such as a, b, c, and P with creep speed (Benkelman beam) deflection, δ, for various times of the year were obtained from the AASHO Road Test data and are written as

$$\delta = \bar{\delta} + \delta_A \sin\left[\frac{2\pi}{365}(t - t_0)\right]$$

where t is the time in calendar days, t_0 is a time parameter, and $\bar{\delta}$ as well as δ_A are functions of a, b, c, and P. Creep speed deflections are also associated with slope variance, rut depth, and roughness.

Results of the Hybla Valley test program are used to correlate physical measure-

ments in the form of rigid plate bearing tests with local factors as well as limited considerations of material properties. Nondimensional parameters are used in relating applied load, strength characteristics of the pavement components, bearing plate size, and pavement component thicknesses to surface, base, and subgrade deflections. Hyperbolic equations are used to represent the response. Correlation of actual results with those given by the equations developed are presented in graphic form.

CHAPTER ONE

INTRODUCTION, RESEARCH OBJECTIVE, AND GENERAL APPROACH

Comfort and convenience may be considered inherent manifestations of pavement performance or degree of serviceability. The performance of a pavement is influenced by many factors, including gross applied load, tire pressure, number of load applications, thickness and durability of the various pavement components, and the mechanical properties of the pavement materials. Thus, functional pavement design should correlate these factors with desired performance characteristics.

At present, most methods of flexible pavement design employ a combination of theoretical and empirical procedures in conjunction with soil classifications, but are not based on performance-serviceability concepts. The present analysis gives behavioral trends that make such an attempt conceptually possible by relating performance as expressed by the present serviceability index (PSI) to various local factors such as surface course thickness, a, base thickness, b, subbase thickness, c, vehicle axle load, P, and number of load applications, N. The general functional form of these behavioral trends may shed valuable insight to possible extrapolation of the AASHO Road Test results to local conditions through satellite test programs.

RESEARCH OBJECTIVE

The objective of the research reported herein is to establish significant trends between pavement response and the various local factors considered. Only flexible pavement systems are considered herein. By necessity, all findings are subject to the conditions assumed in each case. These are explained in detail in later sections of this report.

Although a number of other studies were reviewed, the major portion of the analysis contained herein is based on the performance data obtained from the AASHO Road Test and the rigid bearing plate study conducted at Hybla Valley, Va. The trends established herein have their ultimate application in a more efficient and economical design of flexible pavement sections. A more immediate objective lies in the guidance provided for the design of various satellite test programs to supplement programs conducted to date.

GENERAL APPROACH

One frequent approach to the analysis of such data is based on statistical methods. By use of previous experience or intuition the investigator selects a mathematical model involving the variables under consideration and several constants. On this basis the particular "best fit constants" may be determined by some statistical procedure, such as regression analysis. The success of such an approach depends to a large degree on the ingenuity of the investigator and the particular mathematical model selected in each case. Often times a multiplicity of "constants" results, each applicable to a particular set of circumstances.

The investigation reported herein is based on a phenomenological approach. Such an approach attempts to look more objectively at the actual data and establish phenomenological or observed trends on this basis. The actual data are empirically "fit" with analytical expressions. Resulting "constants" are formulated in terms of the remaining parameters. There is a certain amount of "graphical averaging" inherent in this approach; however, the actual observation of behavioral patterns and selection of analytic expressions provide a very strong point in favor of this approach. Although quite often only one or two typical plots are included, there are a multitude of similar plots which were made prior to deducing any trend. In addition to the merit of such an approach in developing analytic expressions for observational trends, it may provide valuable insight in the selection of a mathematical model for more refined statistical analysis. Wherever convenient, dimensionless parameters are utilized in this investigation.

CHAPTER TWO

PERFORMANCE STUDIES

One basic analysis of the AASHO Road Test data reported herein involves the development of a relationship among the present serviceability index, vehicle axle load, number of load applications, and thickness of the various pavement layers (surface, base, and subbase courses). A second study determines the relationship among creep speed (Benkleman beam) deflection, δ, vehicle axle load, time, and pavement layer thicknesses. Other investigations include relationships between the present serviceability index (PSI) and other performance indices, such as slope variance, *SV*, rut depth, *RD*, and roughness, *R*. The data analyzed were taken from AASHO Road Test data systems 5121-F and 7322-D; further discussion of the details is included in Appendix A. A brief presentation of techniques employed and results achieved is given here. Specific details of each equation developed are given in the appendices.

LOCAL FACTOR RELATIONS

To develop the relationship between present serviceability index (PSI) and local factors, data from approximately 134 flexible pavement sections were first plotted in the form of PSI vs number of load applications, N, the parameters a, b, c, and P being constant for each section. One example of such a plot is presented in Figure 1. It is observed that the response can be approximated by a straight line of constant slope (indicating a rate of deterioration) with jump discontinuities (indicating rapid loss in serviceability) at the spring thaw periods. Other examples of such plots, together with a discussion of the difficulties encountered, are given in the appendices. In general, the equation expressing the relationship may be written

$$\mathrm{PSI}(N) = \mathrm{PSI}_0 - S\,N - \sum_{i=1}^{K} H(N_i)\,(\Delta\mathrm{PSI})_i \qquad (1)$$

in which $\mathrm{PSI}(N)$ is the PSI of a given section after N load applications, PSI_0 is the PSI at $N = 0$, S is the rate of deterioration, $(\Delta\mathrm{PSI})_i$ is the loss in serviceability for the ith spring thaw, $H(N_i)$ is the Heaviside unit step function,

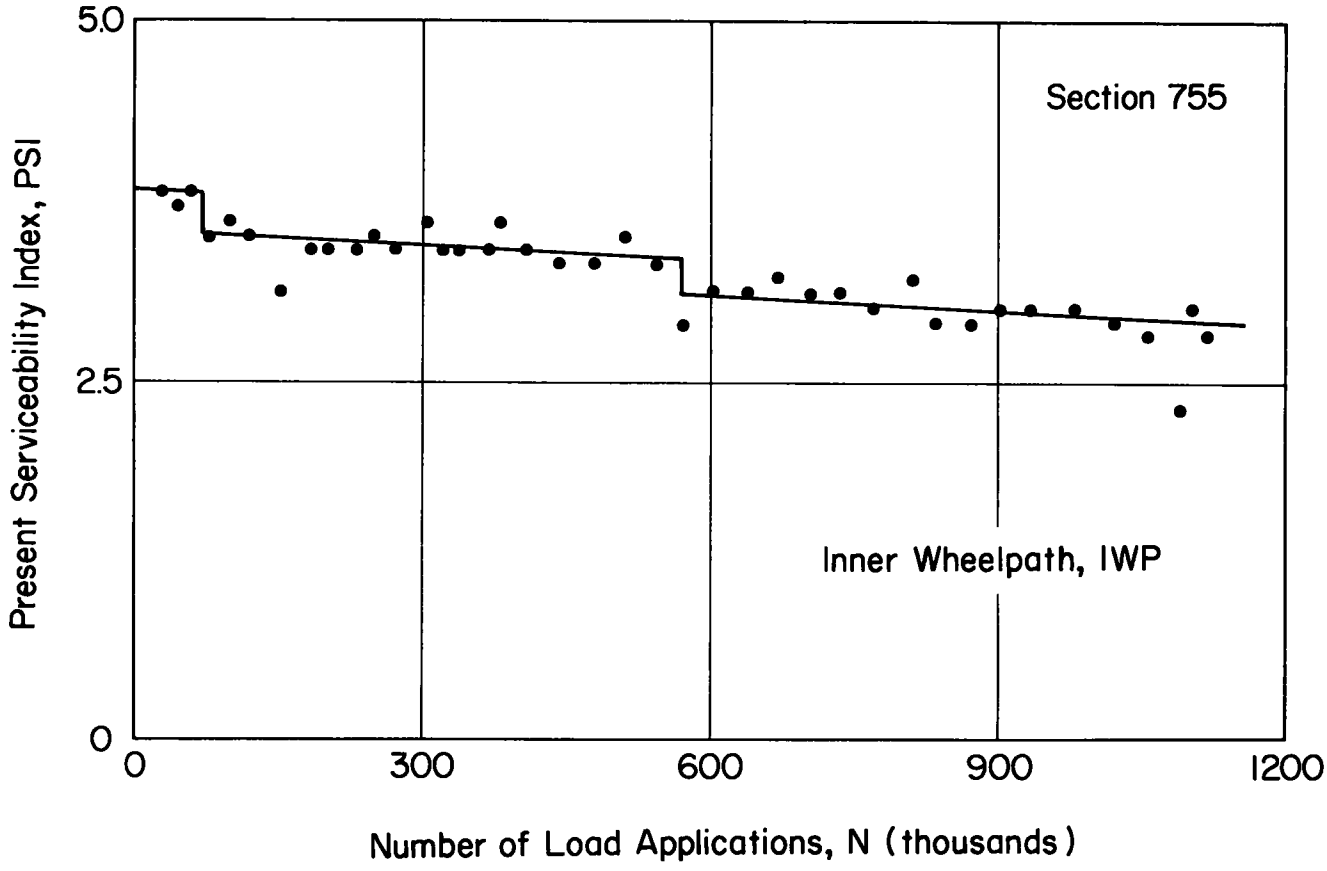

Figure 1. Typical plot of present serviceability index vs number of load applications.

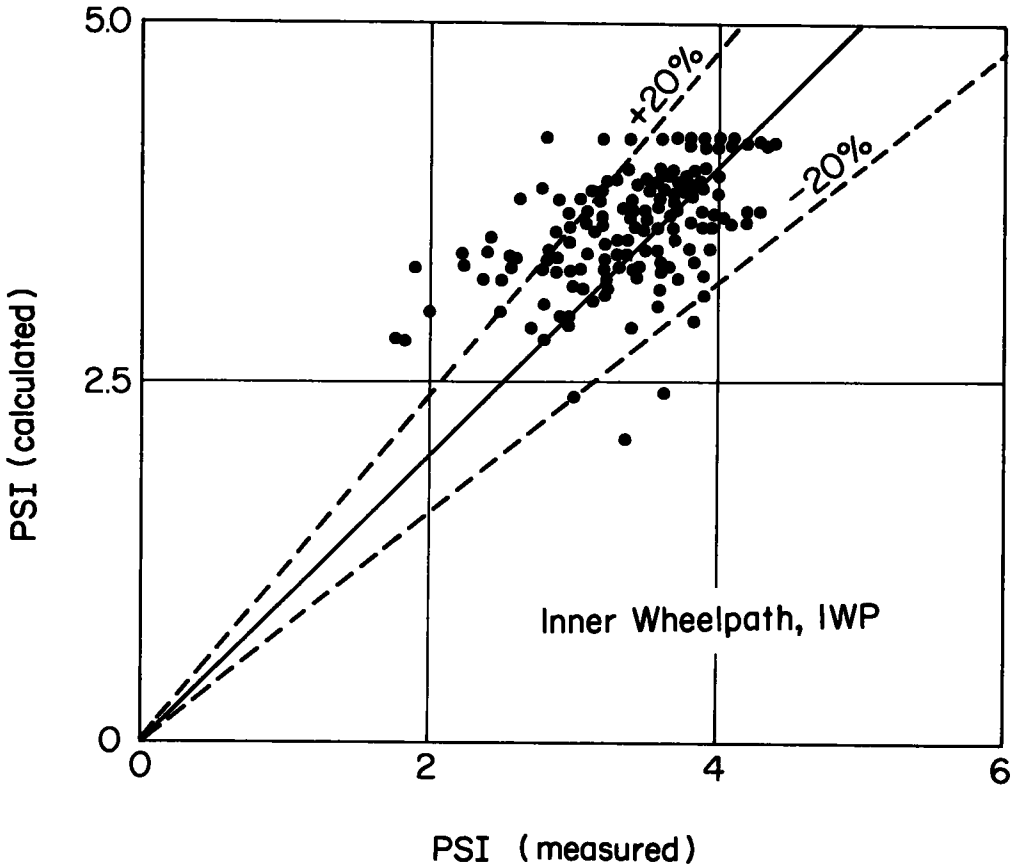

Figure 2. Correlation plot of present serviceability index, Eq. 10.

and K is the number of spring thaws which have occurred during N load applications. Because traffic studies may produce a correlation between N and time, Eq. 1 may be expressed in terms of time. The parameters $(\Delta PSI)_i$ and S will be dependent on a, b, c, and P, while PSI_0 will be a function of construction specifications and quality of workmanship.

As shown in Appendix A, the values for PSI_0 range between 3.3 and 4.6, with the majority of the sections exhibiting an initial PSI of approximately 4.2. Using a value of 4.2 for PSI_0, there is an initial maximum deviation of 21 percent for sections having a PSI_0 of 3.3 before any effects of pavement deterioration are considered. Hence, the value for this term in Eq. 1 should be considered variable, with the recommendation to substitute the actual value obtained. In many cases Eq. 1 will be used to determine the expected change in PSI for a given period of time or a number of applications. For such situations the effect of PSI_0 will cancel out.

To investigate the dependence of S and $(\Delta PSI)_1$ on a, b, c, and P, the results of PSI vs N for the aforementioned 134 test sections were plotted and corresponding values for each parameter were obtained. As explained in Appendix A, the thickness parameters a, b, and c were combined into a characteristic thickness index, D, for each pavement section. The equation utilized has been previously reported (*2*) and may be written

$$D = 0.44a + 0.14b + 0.11c \tag{2}$$

Such a combination of terms reduces the remaining four variables (a, b, c, P) to two variables (D, P). Maintaining P constant, values of S and $(\Delta PSI)_1$—loss in serviceability during the first spring—were plotted versus D for each lane of each traffic loop. These plots are given in Appendix A. Inasmuch as there are ten traffic lanes—each with a different axle loading—ten such plots are obtained for each quantity S and $(\Delta PSI)_1$. From the graphs given in Appendix A, the relationships approximating these data may be written as

$$S = \alpha\, 10^{-\beta D} \tag{3}$$

and

$$(\Delta PSI)_1 = \gamma\, 10^{-\lambda D} \tag{4}$$

where α, β, γ and λ represent coefficients which are dependent on axle loading. Using the equivalency between single-axle and tandem-axle loadings given in Appendix A, these coefficients are each plotted in Appendix A versus P with the resulting general forms for the approximating equations:

$$\alpha = C_1\, 10^{C_2 P - C_3} \tag{5}$$

$$\beta = C_4 + C_5 P \tag{6}$$

$$\gamma = C_6\, 10^{C_7 P - C_8} \tag{7}$$

$$\lambda = C_9 + C_{10} P \tag{8}$$

Substituting Eqs. 5 through 8 in Eqs. 3 and 4, and then substituting these in Eq. 1, gives

$$\mathrm{PSI}(N) = \mathrm{PSI}_0 - C_1\, 10^{C_2P-C_3}\, 10^{-(C_4+C_5P)D}\, N - H(140{,}000)C_6\, 10^{C_7P-C_8}\, 10^{-(C_9+C_{10}P)D} \quad (9)$$

as an expression for determining the PSI of a flexible pavement section. It must be emphasized that this development is limited to the first spring thaw of the AASHO Road Test, which occurred at approximately 140,000 load applications.

The preceding general form of serviceability-local factors relationship is based on observed data from the AASHO Road Test. It is reasonable, however, that the same or a similar though slightly modified form may be employed to describe other data from different projects in different localities. Quantitative values for the coefficients C_1 through C_{10} in Eq. 9 were obtained from the AASHO Road Test data. These are developed in detail in Appendix A and allow Eq. 9 to be written

$$\mathrm{PSI}(N) = \mathrm{PSI}_0 - 0.5(10)^{0.078P}\,[N(10)^{-[(0.35+0.005P)D+6]} + H(140{,}000)(10)^{-(0.24+0.008P)D}] \quad (10)$$

In Eq. 10, $H(140{,}000)$ is the Heaviside unit step function and has the value zero for $N < 140{,}000$ (that is, before the first spring thaw), and unity for $N \geqq 140{,}000$.

For correlation purposes PSI_0 may be taken as 4.2, and values of PSI for N = 50,000, 200,000 and 500,000 may be calculated from Eq. 10 for all in-test flexible pavement sections from all loops. Figure 2 gives a correlation plot between these calculated PSI values and the recorded values for corresponding load applications, as given in AASHO Road Test data system 7322-D. It can be seen that the calculated values correspond to the recorded values within approximately ± 20 percent for most sections. Alternately, in order to cancel the effect of PSI_0, a correlation between the calculated change in PSI and the recorded change in PSI for different number of load applications can be made. Such a plot for N = 50,000, 200,000 and 500,000 is given in Figure 3.

The effect of the Illinois (AASHO Road Test) environment for the first spring thaw has been taken into account in the parameter $(\Delta\mathrm{PSI})_1$, which is given by Eq. 4. It may thereby be considered as a form of "climatic" or regional parameter which represents the annual adversity undergone by the pavement. For practical purposes some estimate of this parameter must be made, but such an estimate is difficult because spring thaws vary from year to year and from location to location. Observations of $(\Delta\mathrm{PSI})_2$ presented in Appendix A indicate that the serviceability loss during the second spring thaw is nearly the same as during the first spring thaw. Extrapolation to future spring thaws and to other climatic regions would require information of the type provided by a comprehensive satellite test program.

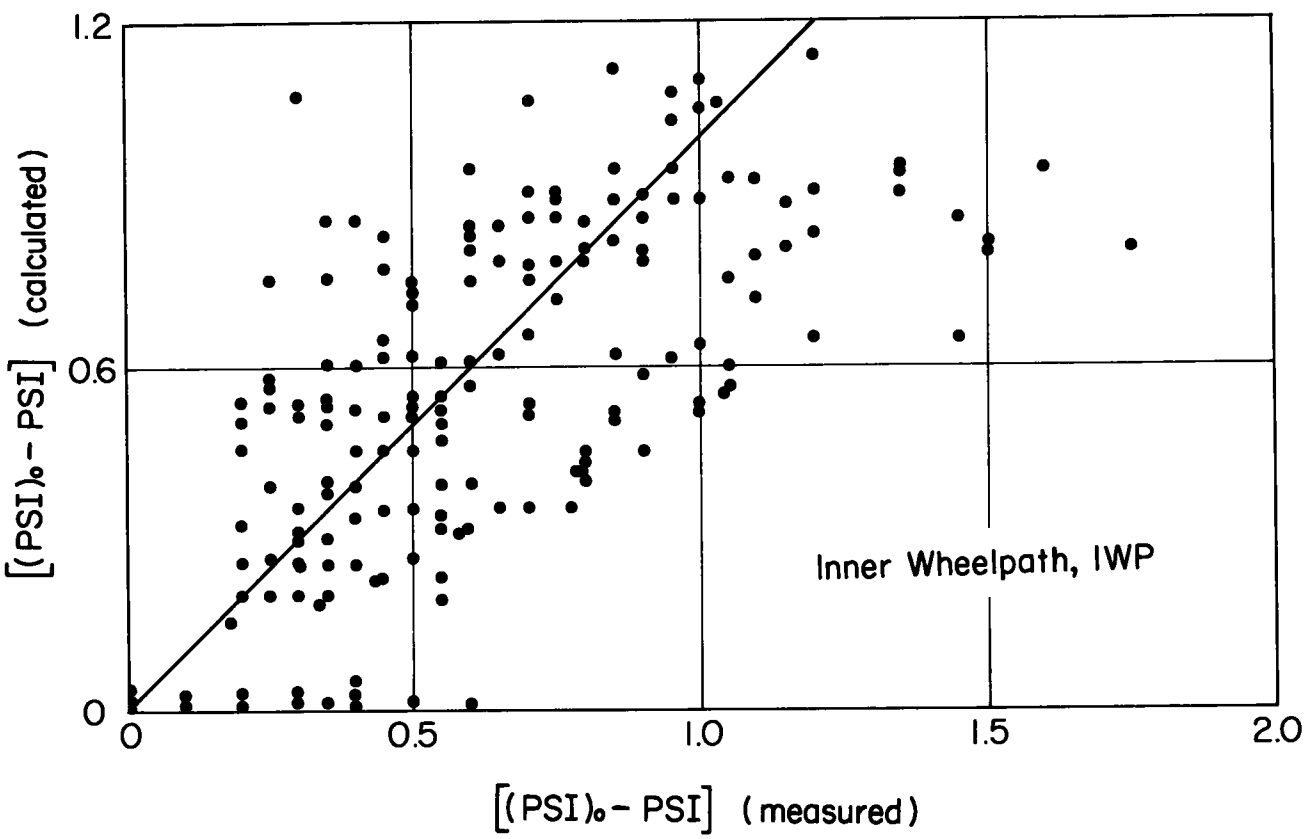

Figure 3. Correlation plot of change in present serviceability index, Eq. 10.

PERFORMANCE-DESIGN CONSIDERATIONS

The foregoing equation for PSI as a function of number of years in service gives highway performance directly in terms of the local design factors (such as the thicknesses of the components of the flexible pavement system and axle load), whereas the equation previously developed by the AASHO Road Test staff gives PSI only in terms of physical measurements (such as slope variance, rut depth, and cracking plus patching) expressing a state of structural deterioration after the highway has been built and in use. Thus, the present developments present the idea of determining PSI before construction from the design variables. This is an important step forward because it advances the concept of specifying performance as a design criterion and selecting a flexible pavement system design to meet the performance criterion. However, the numerical coefficients should be specified in accordance with local climatic and soil conditions.

The following is presented to illustrate the possibility of incorporating serviceability index into pavement design procedure through various local factors of the structural system of the flexible pavement unit and magnitude and number of load applications. The performance level of a given flexible pavement under a given vehicle axle load can be estimated for a given number of load applications, or the pavement can be so designed as to yield a given serviceability index after a given number of load applications. The general form of Eq. 9 can be solved for the thickness index, D, and used as a design equation provided the coefficients C_1 through C_{10} are known. In general, these coefficients would have to be obtained for each specific locality. In addition, the mechanical properties of the materials used would have to be taken into account. Each of these two restrictions introduces serious limitations on the extension of these developed relationships to other conditions. Likewise, such restrictions emphasize the necessity of satellite test programs designed to provide estimates for these coefficients.

For design purposes the equation may be used as follows. Given the initial and final performance limits for a given number of load applications, Eq. 9 is solved for the thickness index, D. Once D is determined, the specific values for each component layer thickness may be found from Eq. 2 by selecting values for any two of the thicknesses a, b, or c and computing the third. Alternately, an optimum design on the basis of minimizing cost can be obtained by a linear programming analysis. Thus, it is possible to have various designs associated with various combinations of a, b, and c that will give the same performance.

For the material properties and environmental conditions of the AASHO Road Test, values for the coefficients have been determined and are substituted into the general form of Eq. 9 to yield Eq. 10. The application of Eq. 10 as a design criterion can best be understood by hypothetical examples. It must be realized that reliable extrapolation of the developed equations must be supplemented by quantitative verification by satellite test programs.

EXAMPLE 1

It is intended to construct a flexible pavement that is similar in all respects to the AASHO Road Test pavement sections, but located in an area where the effects of spring thaws are negligible. Traffic studies show that pavement loading may be approximated by a 30-kip single-axle equivalent load passing every 5 min. Based on an initial PSI of 4.0, it is specified that the pavement should not drop below a PSI of 2.0 in ten years.

Because the pavement is not affected by spring thaws, the relationship as given by the general equation can be written

$$\mathrm{PSI}(N) = PSI_0 - C_1\,(10)^{C_2P-C_3}\,(10)^{-(C_4+C_5P)D}\,N \qquad (11)$$

At the stated rate, the total number of load applications, N, during this period is 1,055,000. Thus, the following can be written

$$2.0 = 1.055\,C_1(10)^{C_2P-C_{3+6}}\,(10)^{-(C_4+C_5P)D} \qquad (12)$$

Substituting the values of C_1 through C_5 as determined from the AASHO Road Test and specializing the equation for a 30-kip load, the thickness index, D, is found to be 3.52 in. From this value the thicknesses of flexible pavement layers can be estimated using Eq. 2. For example, it is found that a structural section of 3-6-12 is satisfactory for the ten years of traffic operation of a 30-kip single-axle equivalent load.

EXAMPLE 2

It is assumed that a flexible pavement satisfying the same conditions as Example 1 is to be constructed in an area where it will be subjected to spring thaw effects. Satellite test programs have estimated the average change in PSI for each spring thaw to be 0.12. Then Eq. 9 can be written

$$D = -\frac{1}{C_4 + C_5P} \times \log_{10} \frac{[\mathrm{PSI}_0 - \mathrm{PSI}(N)] - \sum_{i=1}^{K} (\Delta\mathrm{PSI})_i}{C_1(10)^{C_2P-C_3}\,N} \qquad (13)$$

Substituting the values of C_1 through C_5 from the AASHO Road Test, together with the given performance limits for PSI_0 and $\mathrm{PSI}(N)$ and the number of load applications, N, the thickness index, D, is determined as 4.32 in. The thicknesses for the pavement section can be selected as in Example 1. One possible combination is 5-9-8. The design could be optimized using linear programming techniques.

These two examples illustrate a potential design use for the relationship developed from an analysis of the AASHO Road Test results.

PHYSICAL MEASUREMENTS CORRELATIONS

The foregoing developments as given by Eq. 9 relate the pavement performance as expressed by PSI to such local factors as applied load, number of load applications, and various thicknesses of the structural components comprising the flexible pavement section.

To relate performance of a pavement at any time in its

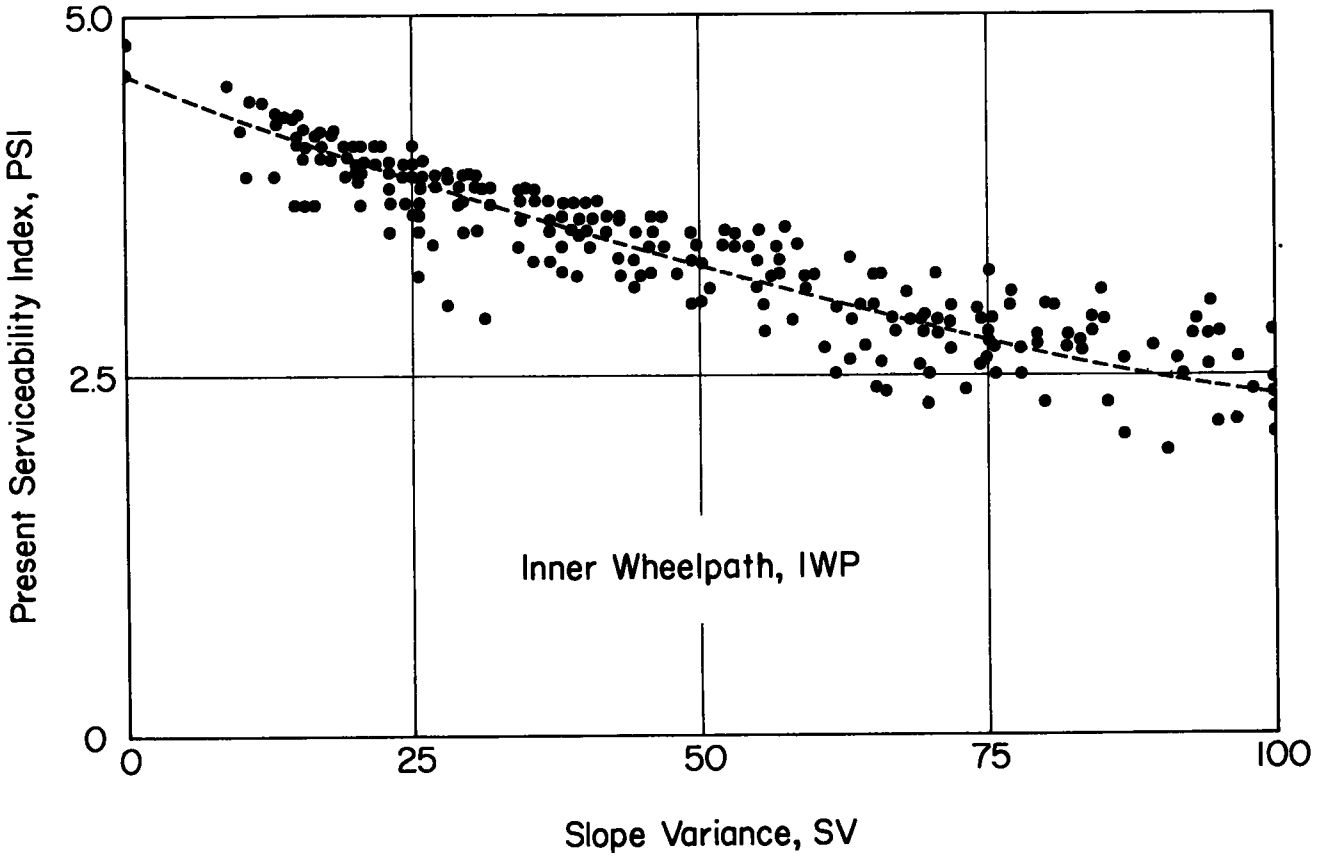

Figure 4. Relationship of present serviceability index and slope variance.

life to measureable quantities, Carey and Irick (*3*) developed the present serviceability index in terms of slope variance, *SV*, rut depth, *RD*, and cracking plus patching, $C + P$. In their development, Carey and Irick assumed certain linearizing transformations for *SV*, *RD*, and $C + P$; th n, values for the weighting coefficients were obtained by regression analysis to give

$$\mathrm{PSI} = 5.03 - 1.91 \log (1 + SV) - 0.01 (C + P)^{\frac{1}{2}} - 1.38 (RD)^2 \quad (14)$$

in which the physical measurements *SV*, *RD*, and $C + P$ are described in Appendix A.

The individual correlations between PSI and these physical measurements were obtained from the AASHO Road Test data and are presented herein for possible combination into general expressions with different weighting techniques than those used by Carey and Irick.

Correlation with Slope Variance

Slope variance constitutes one of the principal factors for structural deterioration of flexible pavements. Plotting PSI versus slope variance, *SV*, gives a relationship which is quite unrelated to the local factors considered (that is, *a*, *b*, *c*, *P*, and *N*). Figure 4 shows such a plot, including data from a variety of structural sections for a variety of different axle loads. All of these data can be reasonably approximated by

$$\mathrm{PSI} = 0.6 + 4.0(10)^{-0.0035\,SV} \quad (15)$$

which is shown as the dashed line in Figure 4.

The band seems to widen as *SV* increases or as PSI drops below approximately 3.0; hence, correlation does not appear quite as good as the pavement section approaches failure. Also, it is noted that there is some slope variance present even for newly constructed sections at the beginning of test traffic.

Correlation with Rut Depth

Rut depth, *RD*, is an equally important index of structural deterioration affecting flexible pavement performance and serviceability. For every section at the AASHO Road Test the rut depth was obtained by direct measurement in each wheel path at 25-ft intervals.

Plotting PSI versus *RD* also indicates a relationship independent of the structural composition of the pavement section and the applied load. Figure 5 shows the results of several such sections. The relationship can be approximated by

$$\mathrm{PSI} = 4.0 - 2.0\,RD \quad (16)$$

which is represented by the dashed line in Figure 5.

The correlations obtained in Figures 4 and 5 for the AASHO Road Test data between PSI and slope variance and rut depth, respectively, are to be expected inasmuch as

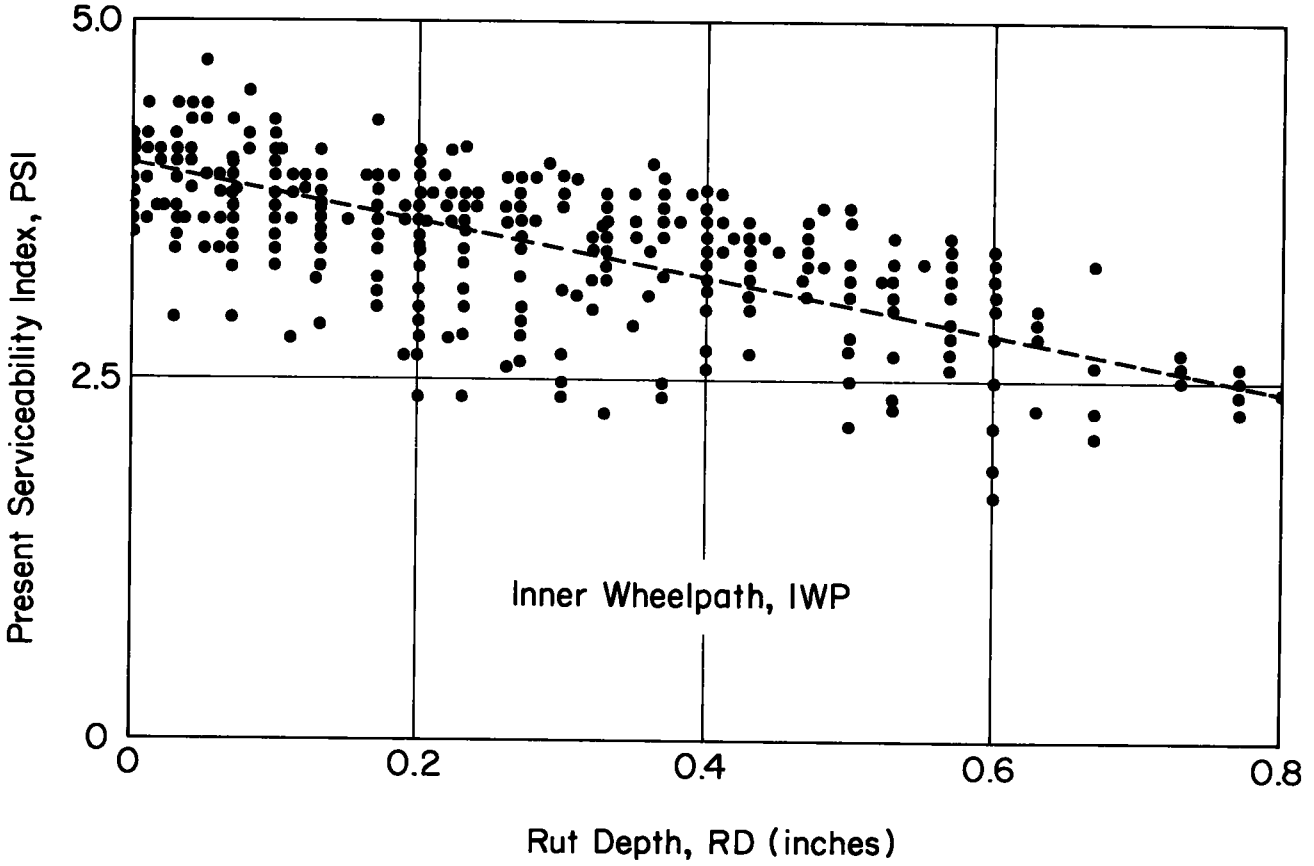

Figure 5. Relationship of present serviceability index and rut depth

both slope variance and rut depth are physical parameters indicative of a form of highway distress and, hence, related to performance characteristics as measured by the present serviceability rating (PSR), which is considered synonymous with PSI in the present report. (See Chapter Three and Appendix A for additional discussion.)

Correlation with Roughness

The roughness index, R, does not seem to exert a strong influence in the determination of PSI values. Figure 6 shows a plot of PSI versus roughness for a number of different sections and axle loads; the relation is reasonably independent of pavement structural composition and applied load and seems to remain relatively constant until $R = 100$, after which it exhibits a decline. An approximate relationship may be written

$$\mathrm{PSI} = 3.7 + H(100)[3.0 - 0.03R] \qquad (17)$$

which is represented by the dashed line in Figure 6.

Correlation with Cracking Plus Patching

For the data systems utilized, very few data were tabulated for cracking and patching. Hence, no attempt was made to deduce any relationship between PSI and $C + P$.

Dependence of PSI on Other Indices

One possible use for Eqs. 15, 16, and 17 is to combine them into a general expression for PSI. As previously explained, one such relationship has been developed by Carey and Irick (*3*) and is given in Eq. 14; however, this may or may not be the optimum relation.

The relationships among PSI, SV, RD, and R given in Eqs. 15, 16, and 17 may shed insight toward the modification of Eq. 14 by selection of different linearizing transformations. For example, considering the indices SV, RD, and R contributing to the determination of PSI, Eqs. 15, 16, and 17 may be combined to yield

$$\mathrm{PSI} = W_1\,[0.6 + 4.0(10)^{-0.0035SV}] + W_2\,[4.0 - 2.0\,RD] + W_3\,[3.7 + H(100)(3.0 - 0.03R)] \qquad (18)$$

where W_1, W_2, and W_3 are weighting factors. Selection of suitable weighting factors to take into account the influence of each of these indices on flexible pavement performance can be made through a statistical procedure. Also, upon availablity of sufficient data, $C + P$ may also be included in a general equation of the type given by Eq. 18.

CREEP SPEED DEFLECTION-LOCAL FACTOR RELATIONS

The correlations between various local factors and creep speed (Benkelman beam) deflections are also presented

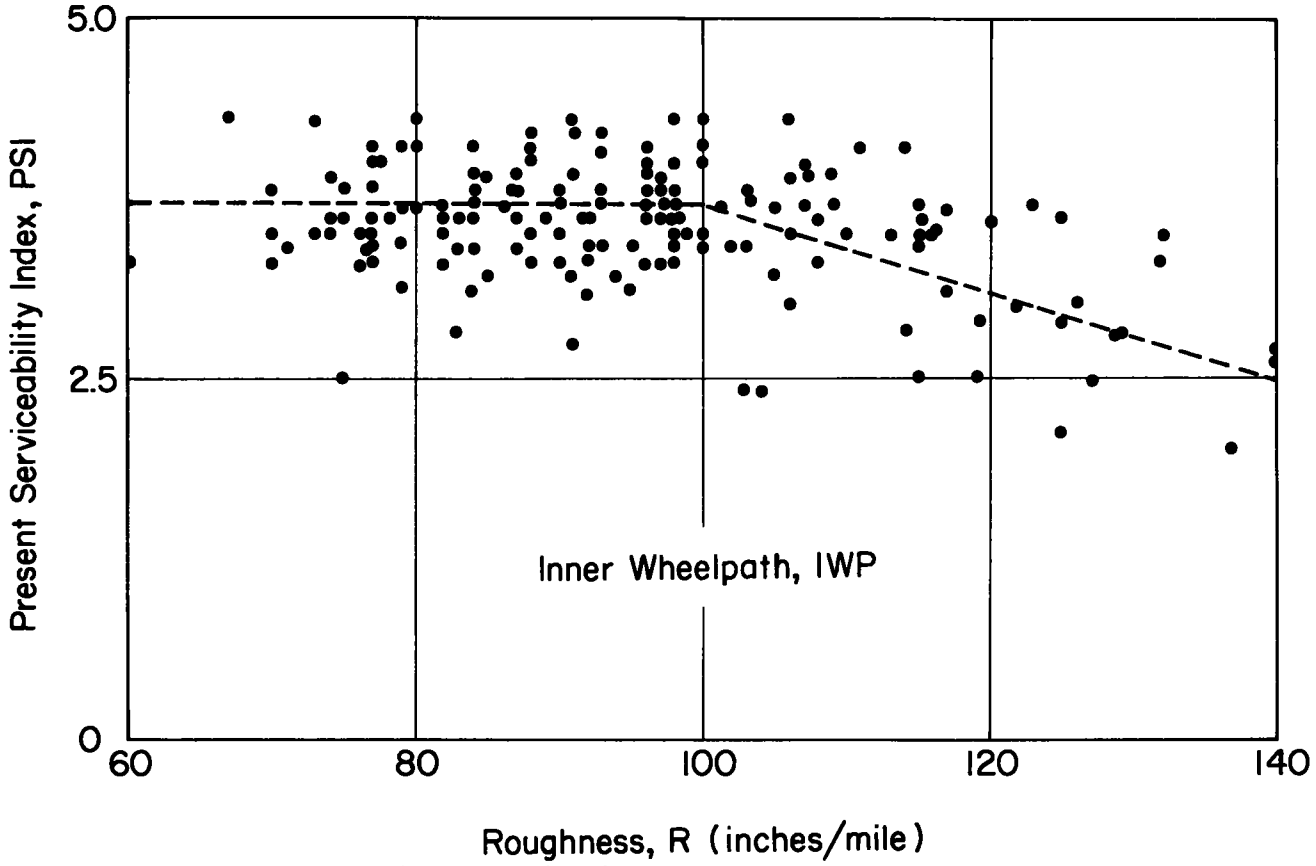

Figure 6. Relationship of present serviceability index and roughness.

herein. Maintaining a, b, c, and P constant, deflection is plotted versus time, and the response is approximated by a sine wave with a period of one year oscillating about a displaced base line, as shown in Figure 7. For convenience, time is used instead of number of load applications. The AASHO Road Test provides a specific correlation between these two parameters; for other situations traffic studies could provide a relationship. Most of the base lines observed from the AASHO Road Test data were very nearly horizontal, with some having a slight tendency toward a gentle negative slope. In addition, the amplitude of the sine wave seemed to remain constant or decrease very slightly for the two cycles observed. For such conditions the approximating response equation for creep speed deflection, δ, can be written as

$$\delta=\bar{\delta}+\delta_A \sin\left(\frac{2\pi}{365}[t-t_0]\right) \tag{19}$$

where $\bar{\delta}$ indicates the level of the base line, δ_A is the amplitude of the deflection wave, t is the time in calendar days measured for the AASHO project, and t_0 is any time in calendar days when the increasing deflection wave crosses the base line. The parameter t_0 may be dependent on the structural thickness of the flexible pavement components due to component material-thickness-temperature-time interrelations which may greatly influence pavement deflection. It is expected that the parameters $\bar{\delta}$ and δ_A will be dependent on a, b, c and P.

Before making an attempt to determine a relationship between δ and the various local factors considered (a, b, c, and P), it is assumed that (a) the base line is horizontal, and (b) the amplitude of oscillation is constant. There is insufficient evidence available for the two-cycle duration of this study to draw any other realistic conclusions regarding these parameters; however, such assumptions are subject to modifications as additional data become available. The next step is to record maximum and minimum deflections (isolated exceptionally high or low deflections were excluded) in lane 1 of Loops 2 through 6 for the two-year duration of the project. The double amplitude, taken as the maximum minus the minimum deflection, is designated $2\delta_A$. The mean deflection, which is the average of the maximum and minimum deflections, is symbolized by $\bar{\delta}$. The mean deflection, of course, establishes the level of the base line.

Utilizing this approach, $\bar{\delta}$ and $2\delta_A$ are each plotted versus the thickness index, D, in Appendix A for each load and approximated by straight lines whose equations in general form are

$$\bar{\delta}=\bar{I}-\bar{S}D \tag{20}$$

and

$$2\delta_A=I_A-S_AD \tag{21}$$

The intercepts and slopes, respectively, of Eqs. 20 and 21 are functions of applied load, P, and these relationships are shown in Appendix A. The following general approximating equations may be written:

$$\bar{I} = C_{11} + C_{12} P \tag{22}$$

$$\bar{S} = C_{13} P \tag{23}$$

$$I_A = C_{14} + C_{15} P \tag{24}$$

$$S_A = C_{16} P \tag{25}$$

where P is expressed in kips.

Substitution of Eqs. 22 through 25 in Eqs. 20 and 21, and then substitution of these in Eq. 19, allows the creep speed deflection-local factor relationship to be written as

$$\delta = (C_{11} + C_{12} P) - C_{13} P D + \tfrac{1}{2}[(C_{14} + C_{15} P) - C_{16} P D] \sin\left(\frac{2\pi}{365}\right)[t - t_0] \tag{26}$$

Such a form accounts for the significantly different values for deflection obtained during the different seasons of the year. As for Eq. 9, the general form of Eq. 26 is based on the AASHO Road Test data, but it is reasonable to expect that the same or a similar functional form would successfully describe other such data. Specific quantitative values as obtained in this analysis of the AASHO Road Test data for the coefficients in Eq. 26 are developed in Appendix A. Utilization of these values allows Eq. 26 to be written

$$\delta = [0.0038 - 0.0006 D]P + \left[\frac{0.005}{P} + 0.00115 - 0.000185 D\right] P \sin\left[\frac{2\pi}{365}(t - t_0)\right] \tag{27}$$

Values of creep speed deflections have been calculated with Eq. 27 for two different seasons of the AASHO Road Test; namely, spring and winter of 1959. Correlation of calculated values of deflection and the corresponding measured deflection values has been made. Figure 8 shows a plot of such a correlation for the spring of 1959; Figure 9 shows a similar plot for the winter of 1959. It may be mentioned that in sketching a measured value of deflection isolated high or low values were neglected. The equation may be expressed in terms of N, utilizing the relation between time and number of load applications.

CREEP SPEED DEFLECTION-PHYSICAL MEASUREMENT CORRELATIONS

Dependence on Rut Depth

A relationship between rut depth and deflection is not readily available. A close observation reveals that RD is not uniquely related to δ alone, but tends to increase with

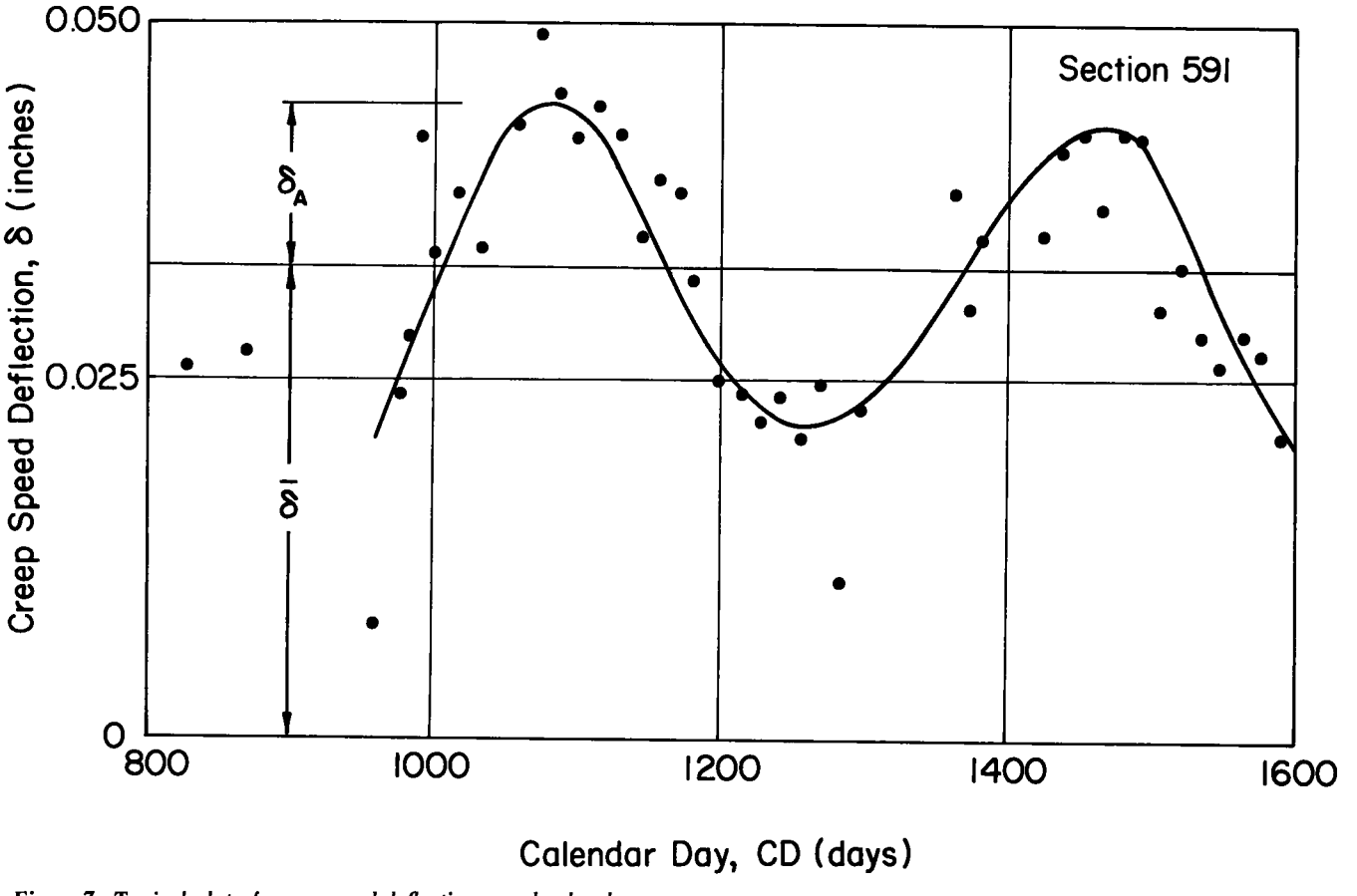

Figure 7. Typical plot of creep speed deflection vs calendar day.

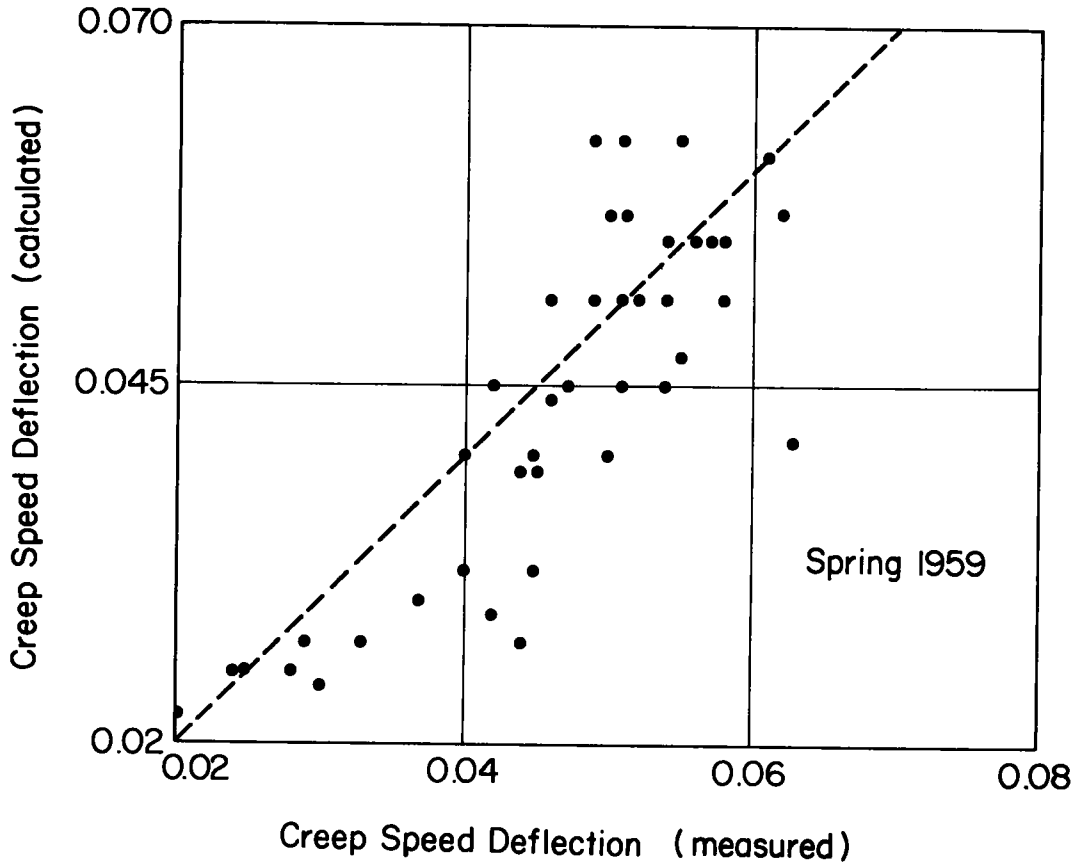

Figure 8. Correlation plot of creep speed deflections, spring 1959, Eq. 27.

increasing N. This suggests selecting a particular relation between RD and δ for constant N. Such a procedure has been followed to a limited degree in Figure 95 of HRB SR 61E(*2*), where creep speed deflections for the spring of 1959 are plotted versus average rut depths after 140,000, 610,000 and 1,114,000 axle load applications. These three sets of data are then fit by three linear equations, as indicated in the figure. This approach can be carried one step farther by taking the intercepts and slopes of these three equations and considering them as functions of N. The slopes may be seen to be nearly constant at 0.062, while the intercept relation is given in Figure 10 and approximated by

$$\text{Intercept} = 0.026 - 0.015 \times 10^{-6} N \qquad (28)$$

Hence, the equation relating δ, RD, and N may be expressed as

$$\delta = 0.026 - 0.015 \times 10^{-6} N + 0.062\, RD \qquad (29)$$

where δ is a special creep speed deflection (namely, that measured in the spring of 1959 under a wheel load equal to the traffic wheel load applied to the loop).

Dependence on Slope Variance

A tendency similar to that pointed out for rut depth exists. For a given deflection the corresponding slope variance tends to increase as N increases.

Dependence on Roughness

There does not seem to be any relation between creep speed deflection and roughness. Roughness does not increase or decrease with the increased number of load applications but simply exhibits a large random scatter pattern. The chances of any significant trends being available are further reduced because the measured roughness data are not available continuously throughout the test duration.

RIGID PLATE DEFLECTION RELATIONS

This study presents an analysis of rigid plates bearing on flexible pavements. The data utilized have been given in tabular form by Benkelman and Williams (*4*) and result from a cooperative endeavor of the Highway Research Board, the Asphalt Institute, and the U.S. Bureau of Public Roads. The field investigation was conducted on a specially constructed track at Hybla Valley near Alexandria, Va.

One of the prime objectives of the field study was to develop, by means of static bearing load tests on full-size pavement sections in the field, fundamental data on the load-supporting value of nonrigid pavement surfacings of various thicknesses in combination with various base course thicknesses and degrees of subgrade support. A description of the test facilities and test procedures is given in Appendix B and in HRB Special Report 46 (*4*).

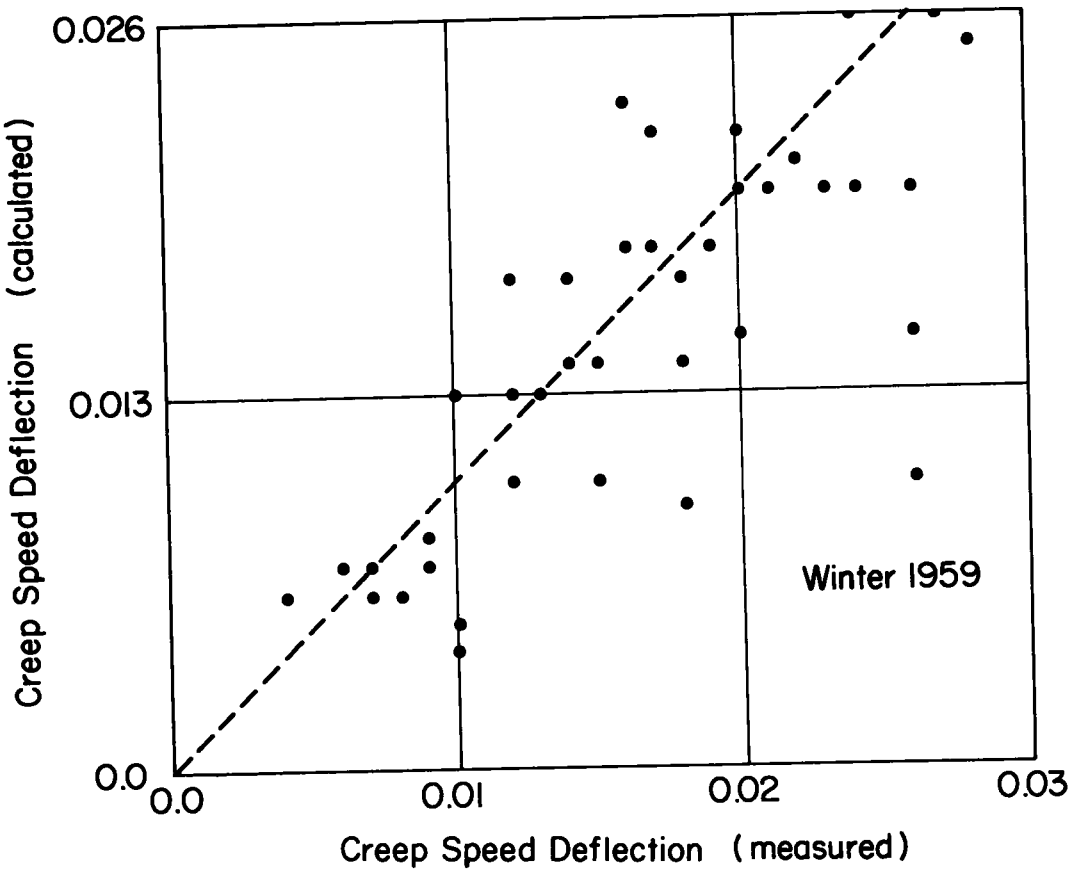

Figure 9. Correlation plot of creep speed deflections, winter 1959, Eq. 27.

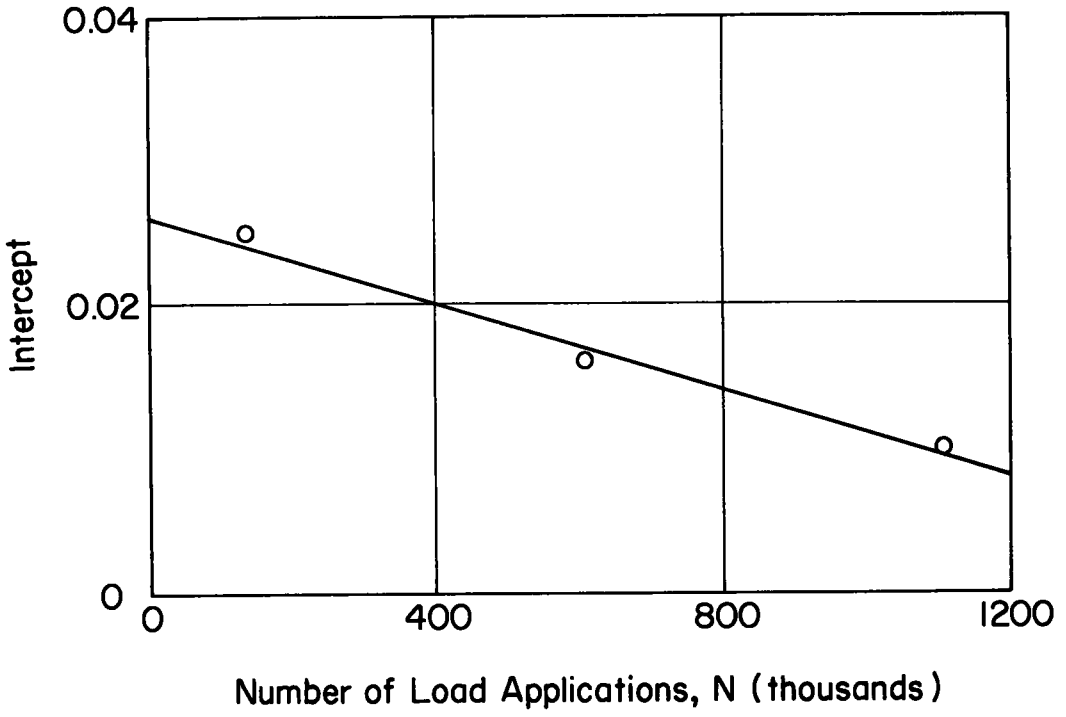

Figure 10. Intercept parameter vs number of load applications; creep speed deflection-rut depth correlation.

Two previous attempts were made by Ingimarsson (*5*) and Kondner and Krizek (*6*) to analyze various portions of the Hybla Valley data as reported in HRB SR 46 (*4*). In addition, Housel (*7*) gives a discussion of Ingimarsson's work and discusses several problems associated with the Hybla Valley data. These works are discussed in Appendix B. This study employs the techniques of dimensional analysis to express the relationships among the variables under consideration. Pursuing the theoretical considerations and restrictions explained in Appendix B, the following functional relationship is obtained:

$$\frac{p}{q} = F\left[\frac{x}{d}, \frac{a}{d}, \frac{b}{d}, N\right] \tag{30}$$

in which x is the deflection, p is the applied pressure, q is the unconfined compressive strength, d is the plate diameter, a is thickness of asphaltic concrete, b is thickness of base course, and N is the number of load applications. For the portions of the analysis which deal with only one load application, Eq. 30 may be reduced to

$$\frac{p}{q} = F\left[\frac{x}{d}, \frac{a}{d}, \frac{b}{d}\right] \tag{31}$$

General Approach

The analysis pursued herein is based on methods of dimensional analysis and is a continuation of work previously reported by Kondner and Krizek (*6*). Three major extensions of their work include: (a) more comprehensive testing procedure and conditions; (b) deflections of component pavement layers; and (c) more general load-deflection relationships. More details concerning the specifics of these extensions are given in Appendix B.

The analytic equation utilized to represent the load-deflection relationship is a two-constant hyperbola in rectangular Cartesian coordinates. In terms of the dimensionless parameters under consideration the equation, as explained in Appendix B, may be written

$$\frac{p}{q_N} = \frac{x/d}{A + B\,x/d} \tag{32}$$

where q_N is a normalized strength parameter. The original data corresponding to each individual test are examined in terms of the foregoing parameters or some appropriate modification. The coefficients A and B are presented in terms of the plate diameter and thicknesses of pavement component layers.

Duplicability

Duplicability of field test results is reviewed in Appendix B and found to be quite poor in many cases. Such inability to duplicate response casts suspicion on all data, especially those for which no attempt at duplication was made. Such divergent results can never be reconciled in any coherent exact analysis and the establishment of trends among the variables must be accepted as an end.

Experimental Results

SUBGRADE

Using data from Table 3 of HRB SR 46, test results from bearing plates 12 to 84 in. in diameter resting on the subgrade were analyzed. For these data the hyperbolic coefficients A and B have been determined in Appendix B to be 0.00018 and 0.0125, respectively. Hence, a general descriptive equation for these data may be written

$$\frac{p}{q} = \frac{x_G/d}{0.00018 + 0.0125\,x_G/d} \tag{33}$$

where x_G represents the subgrade deflection. A correlation plot of pressures calculated by Eq. 33 and measured pressures given in Table 3 is presented in Figure 11 to show the accuracy of Eq. 33 in representing the data.

SUBGRADE PLUS SURFACE COURSE

The data used to develop the relationship for the deflection of subgrade plus surface course, x_P, were obtained from Table 7 of HRB SR 46 and are described in more detail in Appendix B. For this situation the coefficients A and B are functions of the parameter a/d and are found in Appendix B to be explicity expressed as

$$A = 0.00018 + 0.00012\,a/d \tag{34}$$

and

$$B = 0.0125 - \frac{a/d}{12 + 70\,a/d} \tag{35}$$

Using the generalized hyperbolic representation previously developed (Eq. 32), these data may be approximated by

$$\frac{p}{q} = \frac{x_P/d}{0.00018 + 0.00012\,a/d + \left[0.0125 - \dfrac{a/d}{12 + 70\,a/d}\right] x_P/d} \tag{36}$$

The correlation plot (Fig. 12) shows the reliability of the equation to represent the experimental data.

SUBGRADE PLUS SURFACE AND BASE COURSES

Pavement Deflections.—Using the thickness index, D, given by Eq. 2, data from Table 4 of HRB SR 46 were analyzed to determine the dependence of the hyperbolic coefficients A and B on pavement layer thicknesses. Detailed explanations in Appendix B show these relationships to be

$$A = \frac{4.0d^2}{a\,b} \times 10^{(3.33\,D/d)-6} \tag{37}$$

and

$$B = 8.0 \times 10^{(-3.00\,D/d)-3} - 0.001 \tag{38}$$

Substitution of these parameters in the general hyperbolic equation (Eq. 32) gives

$$\frac{p}{q_N} = \frac{x_P/d}{\dfrac{4d^2}{a\,b} \times 10^{(3.33D/d)-6} + [8 \times 10^{(-3D/d)-3} - 0.001]\,x_P/d} \tag{39}$$

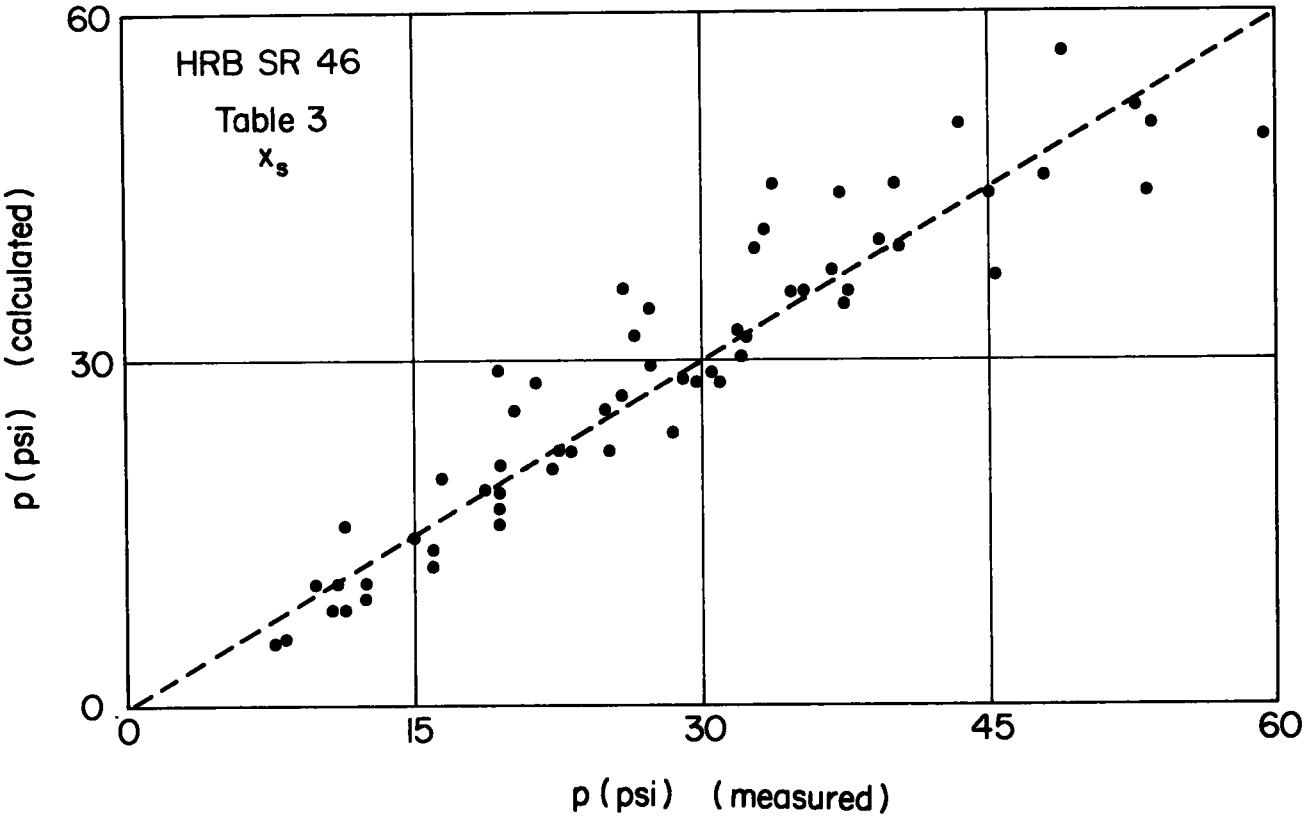

Figure 11. Correlation plot of applied pressures; subgrade deflection, Eq. 33.

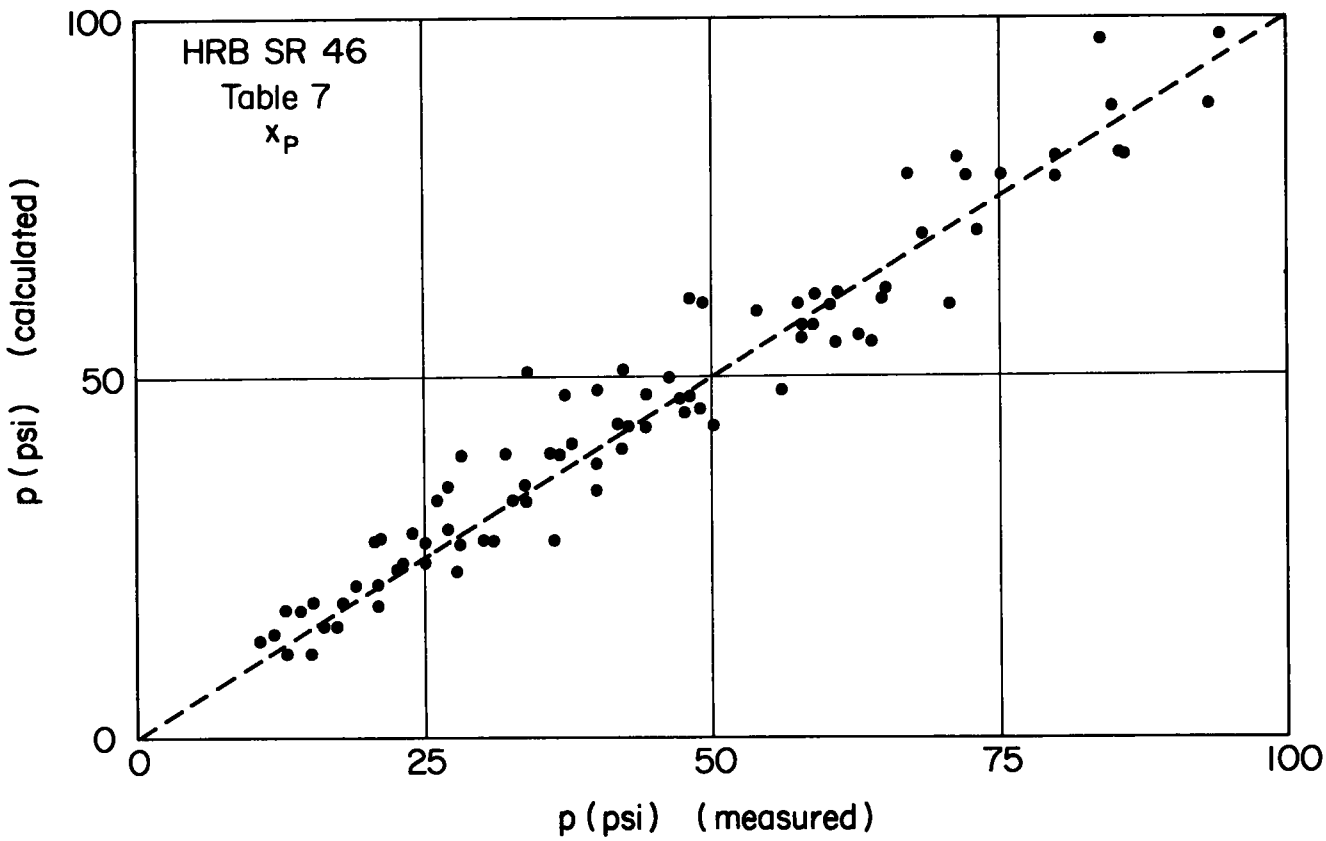

Figure 12. Correlation plot of applied pressures; pavement deflection, Eq. 36.

The correlation between calculated response and measured response can be seen in Figure 13.

Base Course Deflections.—The equation expressing the relationship between base course deflections and applied surface pressures for a rigid bearing plate on the surface is developed in Appendix B for the data from Table 4 of HRB SR 46. In brief, it considers an effective normalizing plate diameter given by $d + 2a$, and by a similar process of determining the dependence of A and B on a, b, and d leads to

$$\frac{p}{q_N} = \left[\frac{d+2a}{d}\right]^2 \frac{x_B/(d+2a)}{\frac{3d^2}{ab}\times 10^{(3.42D/d)-6} + \left[\frac{a}{b}\times 10^{-1-(3.23D/d)}\right]x_B/(d+2a)} \qquad (40)$$

Figure 14 gives a correlation between calculated and measured pressures.

Subgrade Deflections.—Using an effective normalizing diameter given by $(d + 2a + 2b)$, data from Table 4 of HRB SR 46 are analyzed in Appendix B using the same techniques previously described to yield

$$\frac{p}{q_N} = \frac{\left[\frac{d+2a+2b}{d}\right]^2\left[\frac{x_S}{d+2a+2b}\right]}{2.5\frac{D}{d}\times 10^{-5-(2\log D/d)} + \frac{d^2}{ab}\left[3\times 10^{(2.3D/d)-3} - 0.005\right]\frac{x_S}{d+2a+2b}} \qquad (41)$$

The correlation plot (Fig. 15) gives an indication of the reliability of Eq. 41.

TEMPERATURE EFFECT

To obtain some appreciation for the effect of temperature on the various coefficients obtained, data from Table 6 of HRB SR 46 were analyzed (Appendix B). Table 4 and Table 6 data both result from the same testing procedure; however, Table 6 data were obtained at approximately 45 ± 5 F, whereas Table 4 data were measured at 84 ± 6 F. The hyperbolic coefficients A and B are obtained as functions of D/d in Appendix B; substitution of these developed functions in the generalized hyperbolic relation (Eq. 32) yields

$$\frac{p}{q_N} = \frac{x_P/d}{\frac{3d^2}{ab}\times 10^{(2.7D/d)-6} + [12\times 10^{(-2.5D/d)-3}]x_P/d} \qquad (42)$$

The correlation between calculated and measured pressures can be seen in Figure 16. Except for temperature effects, Eq. 42 represents essentially the same test conditions as Eq. 39; hence, the variation in coefficients may be attributed

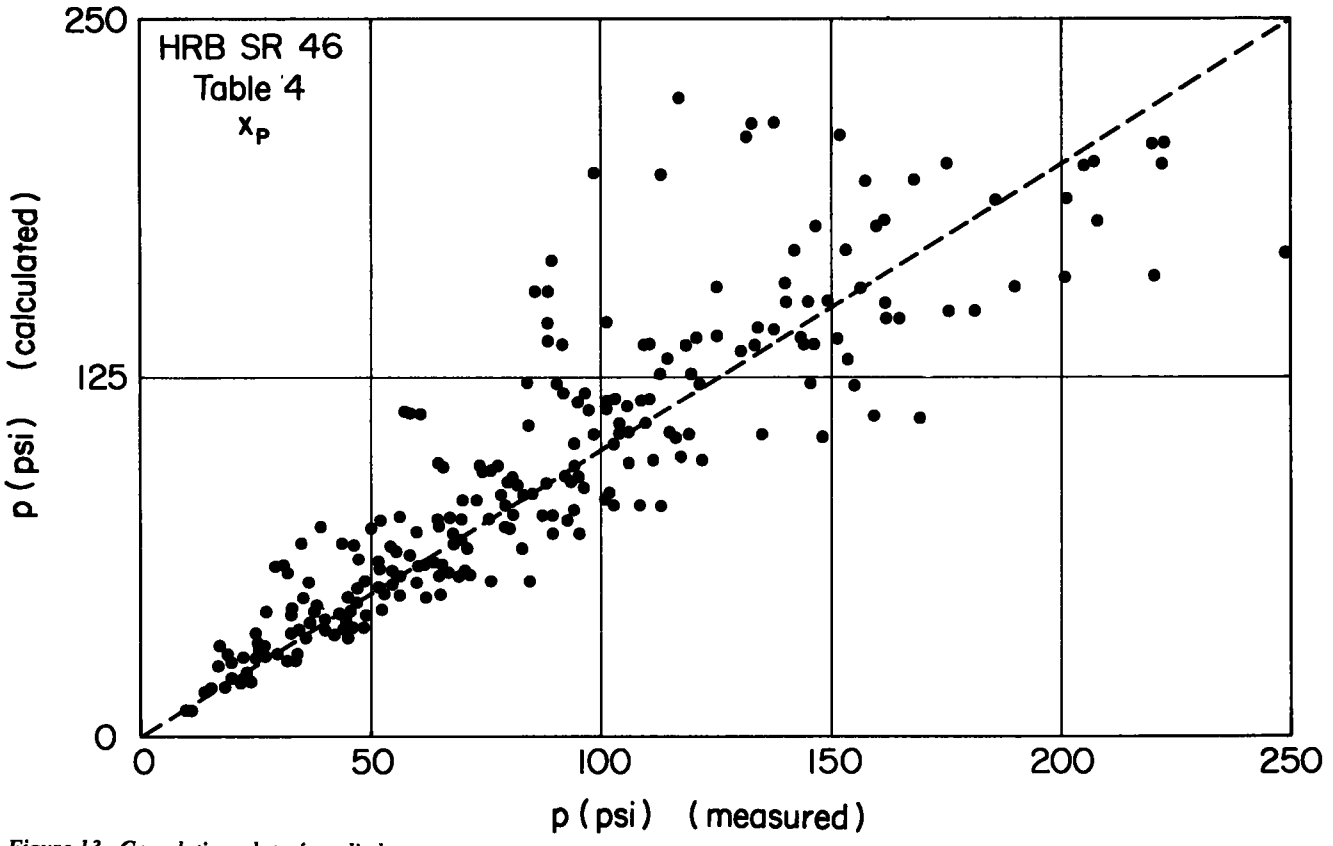

Figure 13. Correlation plot of applied pressures; pavement deflection, Eq. 39.

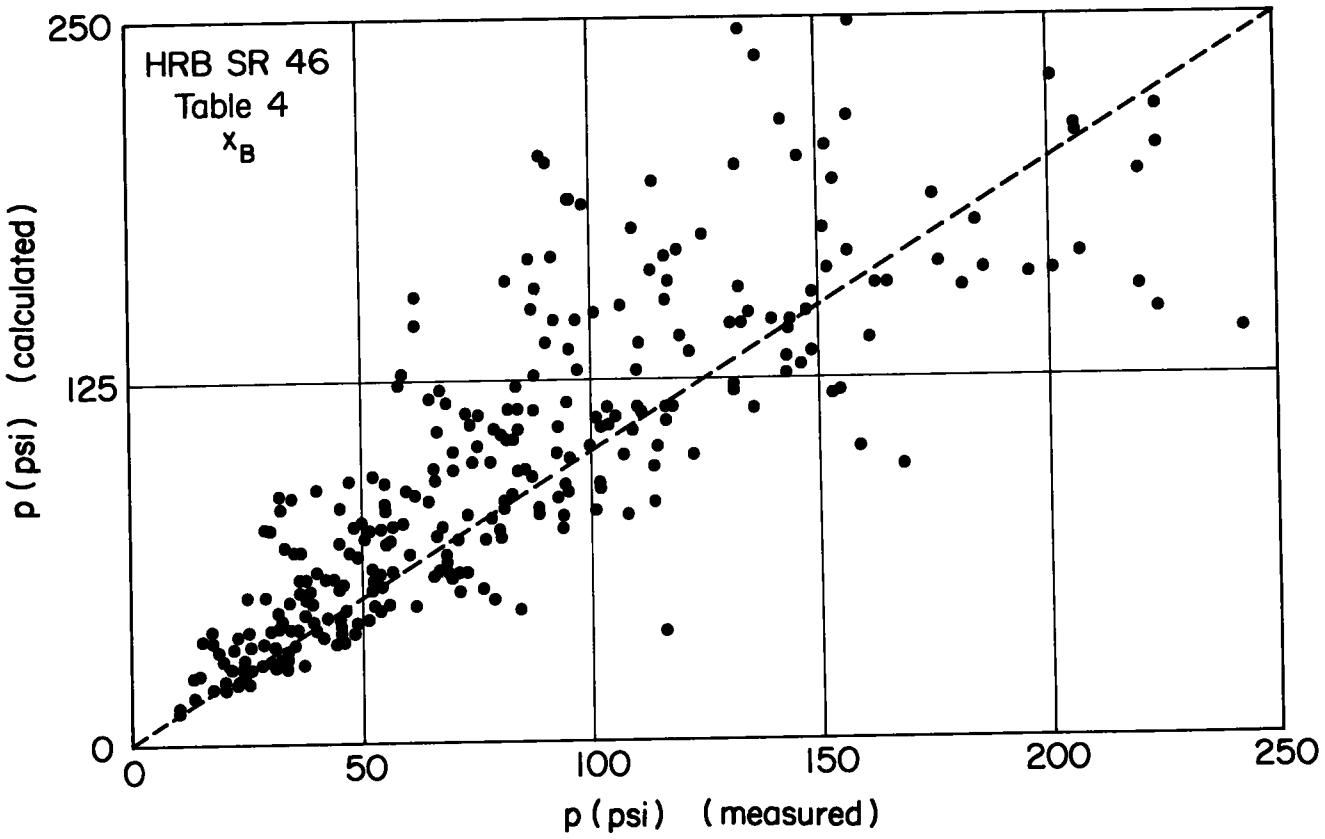

Figure 14. Correlation plot of applied pressures; base course deflection, Eq. 40.

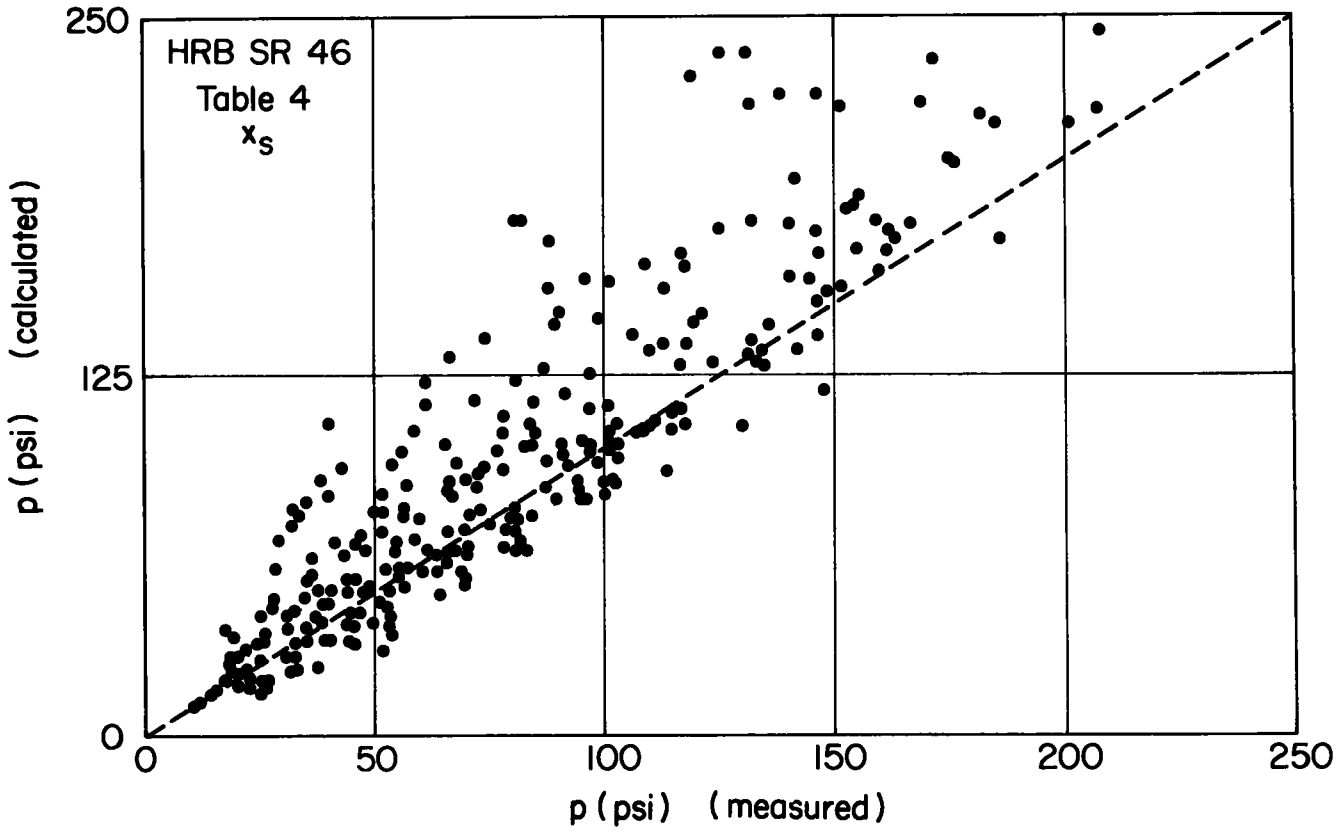

Figure 15. Correlation plot of applied pressures; subgrade deflection, Eq. 41.

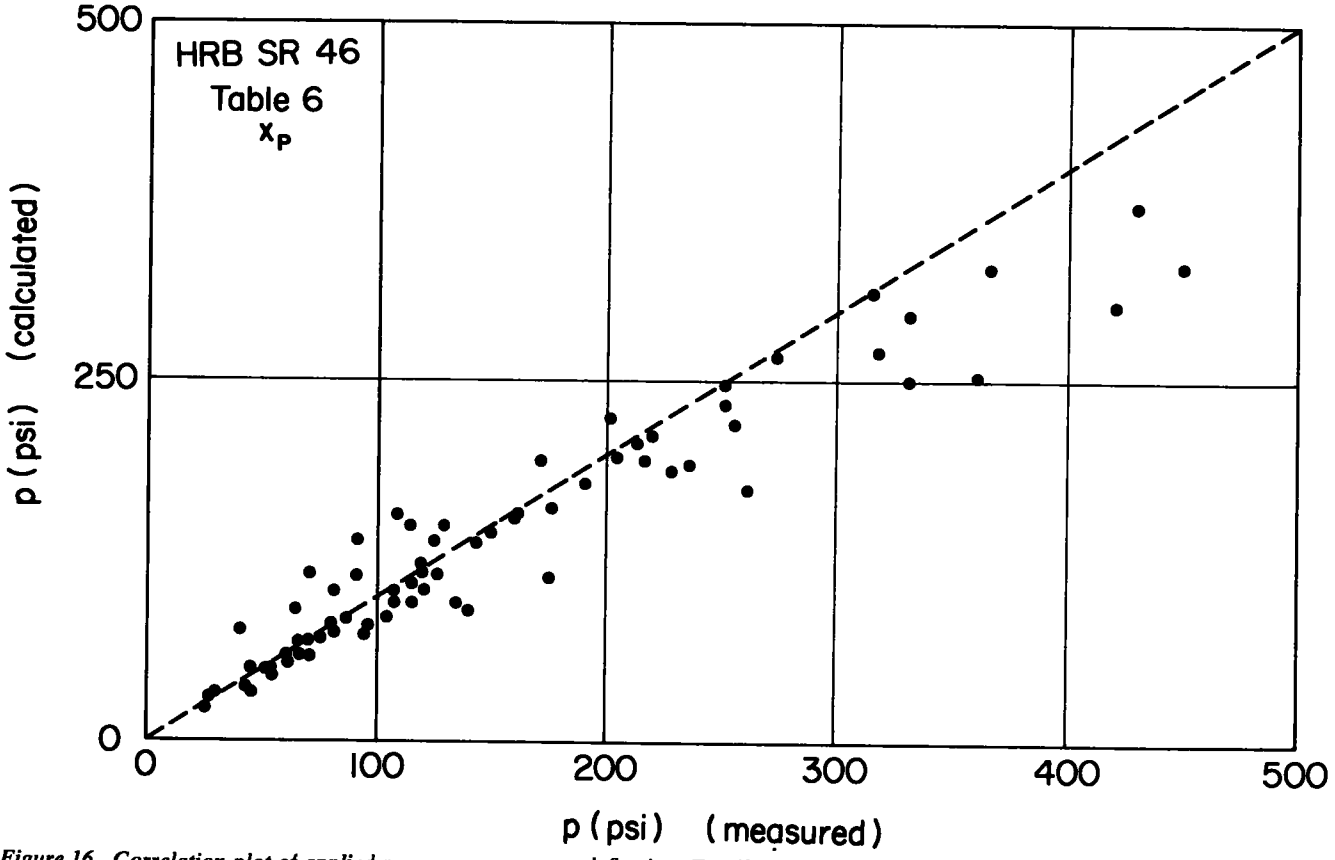

Figure 16. Correlation plot of applied pressures; pavement deflection, Eq. 42.

to this cause. As one example of these effects, for $D/d = 0.3$ Eq. 42 will yield an initial slope approximately twice as great as Eq. 39, thus reflecting the "stiffer" material due to a lower temperature.

REPETITIVE LOADING EFFECTS

The data reported in Table 8a of HRB SR 46 involve a test procedure different from that employed in Tables 4 and 6. These tests were load-controlled (instead of deflection-controlled) and include up to 75 load applications. However, the data associated with one load application were successfully analyzed in a manner similar to the preceding; that is, using the generalized hyperbolic formula and determining the coefficients as functions of the thickness parameter, D/d. Such an analysis, presented in detail in Appendix B, yields

$$\frac{p}{q_N} = \frac{x_P/d}{\frac{1.6d^2}{a\,b} \times 10^{(5.8D/d)-6} + [0.006 - 0.037D/d]\, x_P/d} \quad (43)$$

The differences in the functional relationships for the coefficients are probably attributable to the differences in test procedure. Figure 17 is a correlation plot comparing calculated (Eq. 43 is solved for x_P) and measured deflection values.

To analyze the effect of repetitional loading, the dimensionless factor, f, was defined as the ratio of the deflection at N applications to the deflection at one application. Data were available for 10, 40, and 75 load applications. The general functional relationship for f has been found in Appendix B to be

$$f = 1.0 + \frac{N}{28 + [1.44\, D/d - 0.04]\,(d/a)N} \quad (44)$$

Figure 18 is a correlation plot for calculated and measured values of this factor.

SOIL STRENGTH DATA

As previously explained, because neither soil strength data nor moisture content data for the soil directly under a plate test were available, the soil strength for all tests was assumed constant and results are given in terms of a normalized soil strength ($q_N = 1$). However, data from more than 30 triaxial compression tests on the embankment material have been made available through the courtesy of E. S. Barber. The triaxial tests were conducted with a lateral confining pressure of 1 kip/sq ft; moisture contents ranged from approximately 20 to 32 percent; and wet densities varied from approximately 118 to 128 pcf. The stress-strain response is normalized in terms of the maximum deviator stress, $(\sigma_1 - \sigma_3)_{max}$, and failure strain, ϵ_f, in a manner previously described by Krizek and Kondner (*8*), and the results are given in transformed hyperbolic form in Figure 19. For these tests, a form of Eq. 32 provides a reasonable

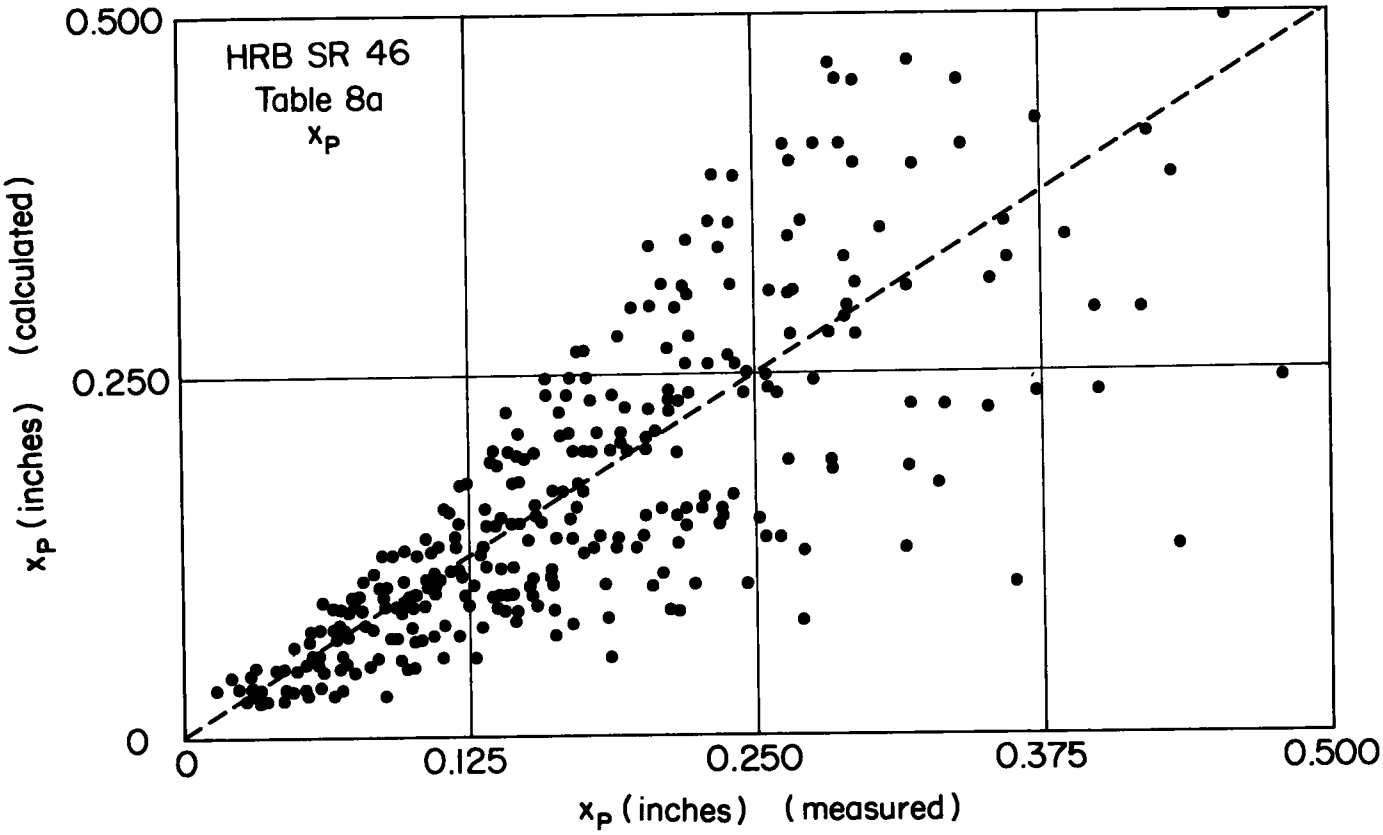

Figure 17. Correlation plot of rigid plate deflections; pavement deflection, Eq. 43.

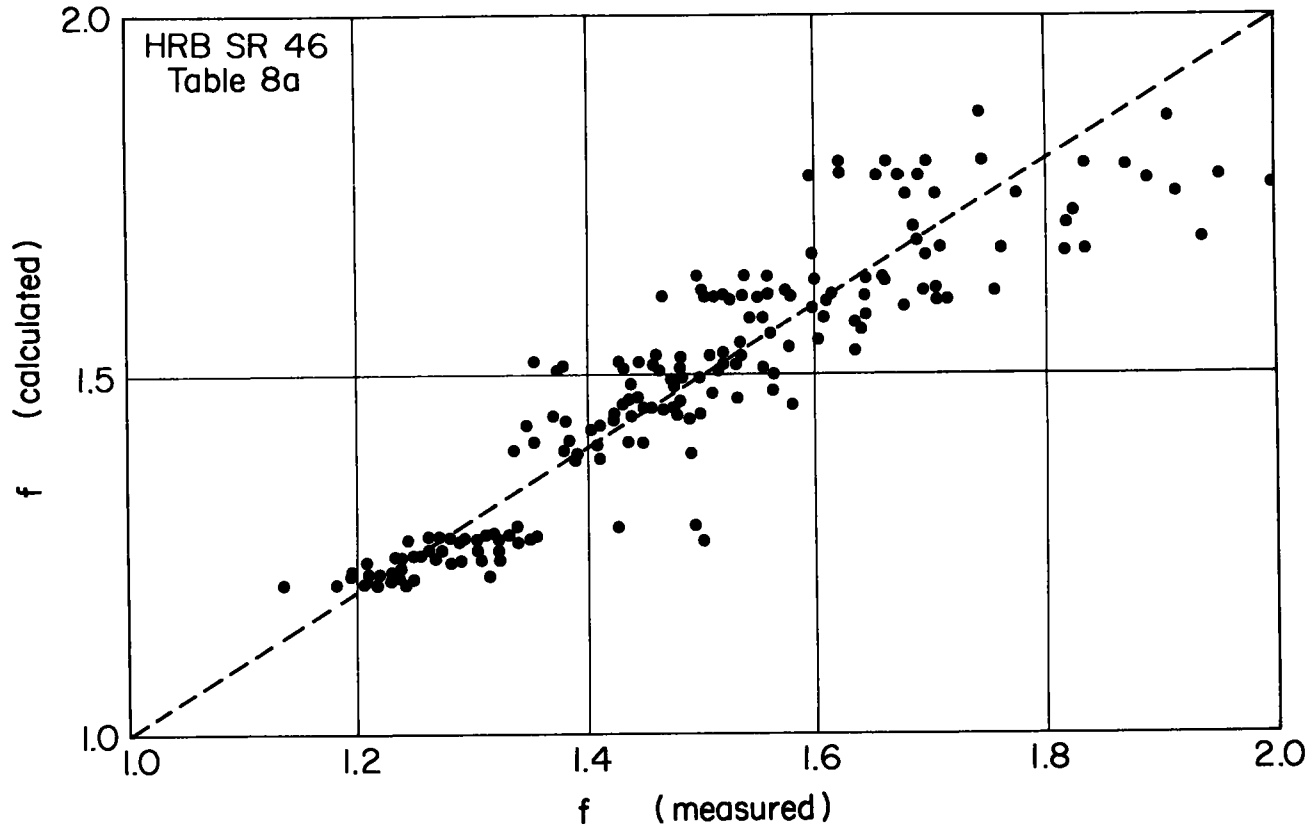

Figure 18. Correlation plot of repetitional deflection factor; pavement deflection, Eq. 44.

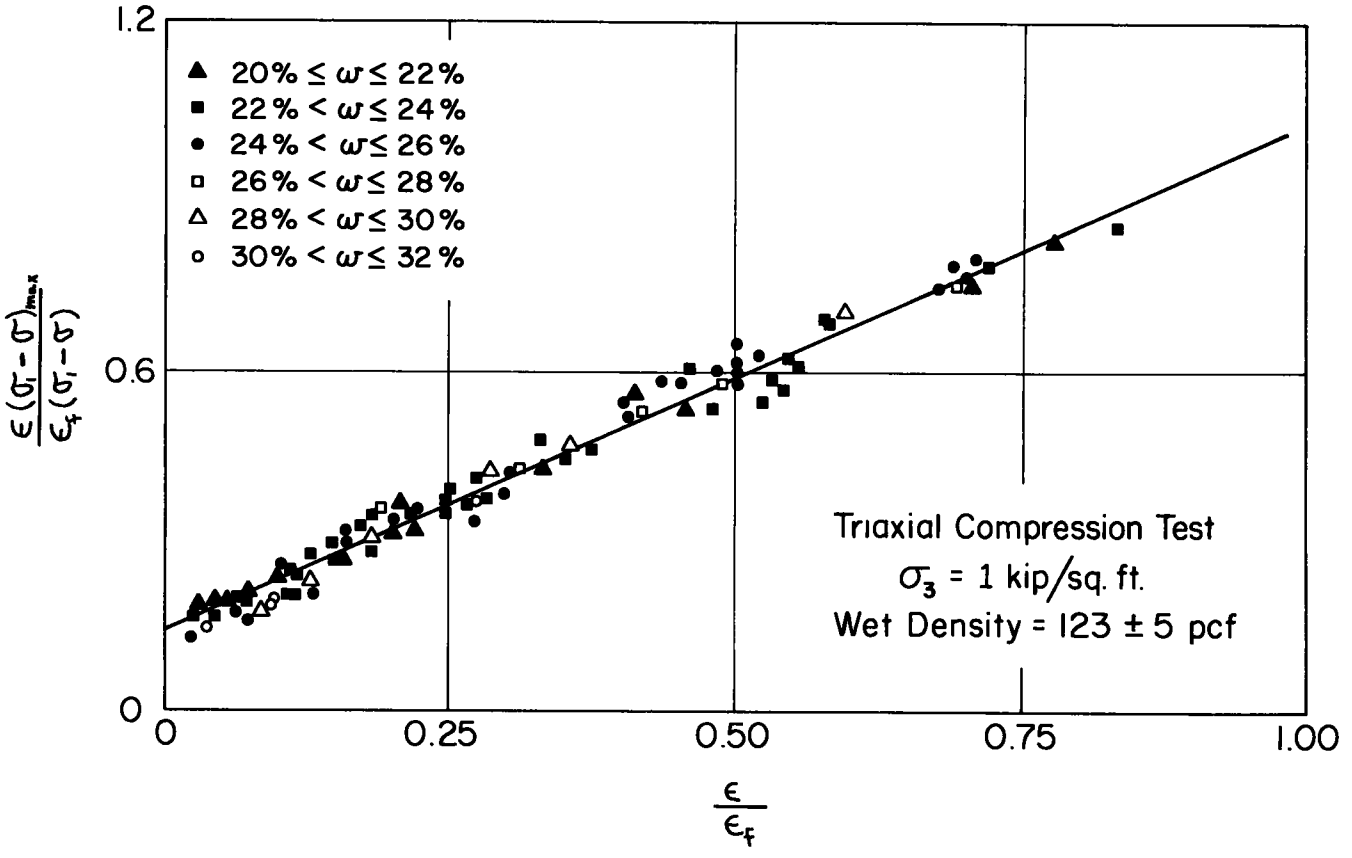

Figure 19. Transformed hyperbolic stress-strain response; triaxial test data.

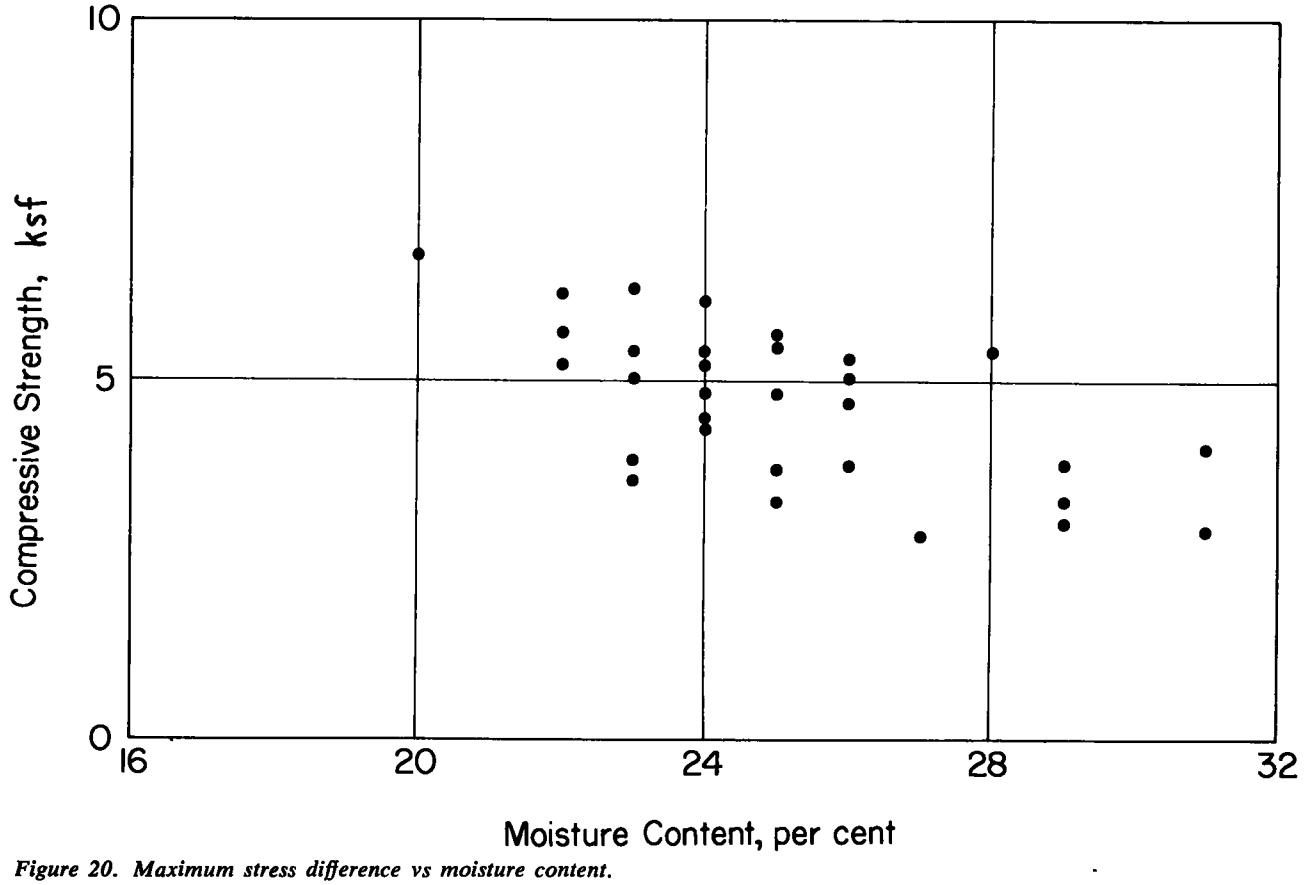

Figure 20. Maximum stress difference vs moisture content.

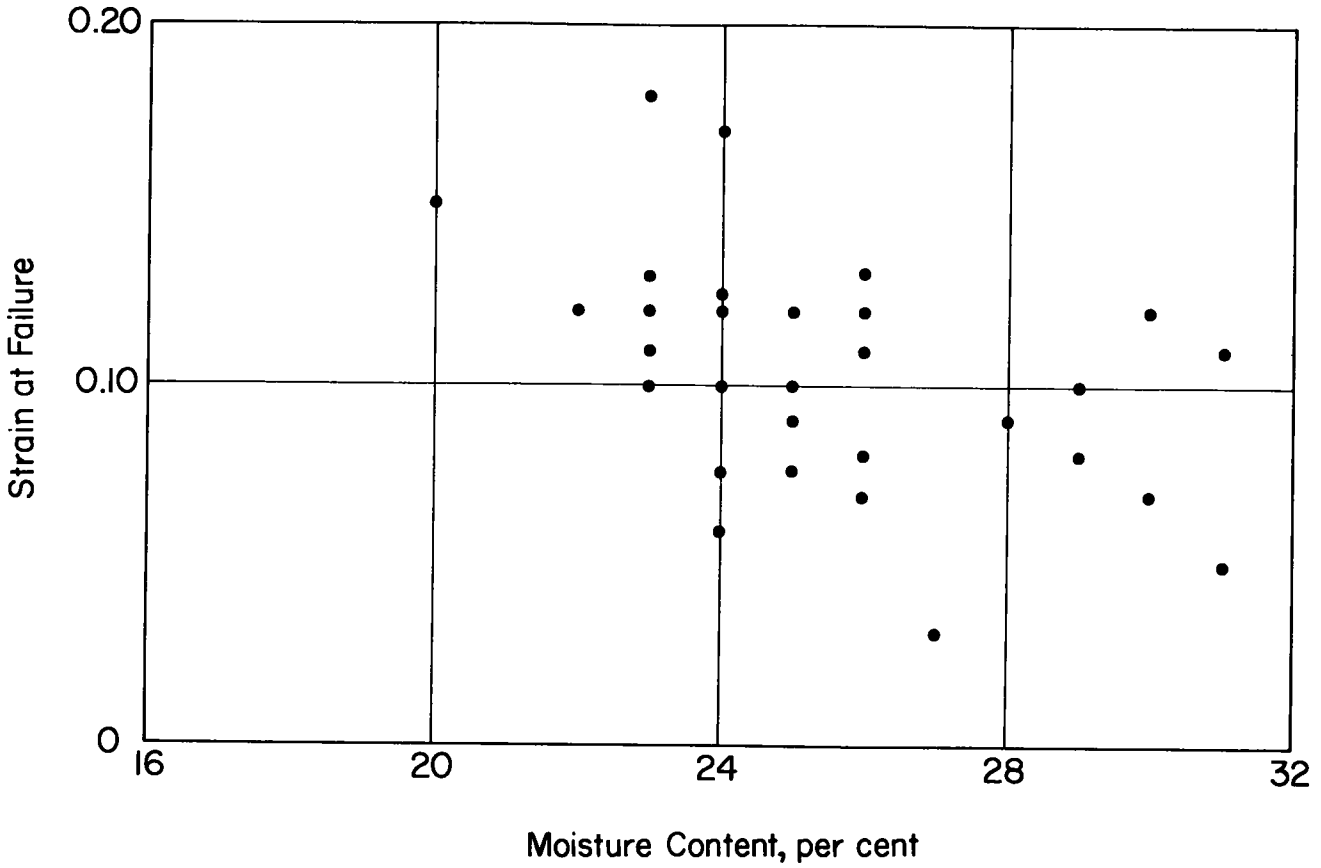

Figure 21. Strain at failure vs moisture content.

approximation for constitutive behavior; this equation may be written as

$$\frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_{max}} = \frac{\epsilon/\epsilon_f}{0.14 + 0.88\,\epsilon/\epsilon_f} \qquad (45)$$

The consistency indices, $(\sigma_1 - \sigma_3)_{max}$ and ϵ_f, are shown as functions of moisture content in Figures 20 and 21. If it were possible to identify the particular test results with a given soil strength, these results could be incorporated in the response equations.

CHAPTER THREE

SUMMARY OF FINDINGS

PERFORMANCE STUDIES

One of the most significant findings presented in this report is the development of a trend between flexible pavement performance and local factors considered, such as pavement structural composition, applied load, and number of load applications. This development makes use of the basic assumption that flexible pavement serviceability may be realistically represented by a present serviceability index (PSI) which has been correlated with a panel serviceability rating. The selected expression used to describe PSI as a function of number of applications, N, assumes that the rate of deterioration (that is, $\Delta PSI/N$) is constant during both years of the controlled traffic phase of the AASHO Road Test and that the very rapid deterioration associated with spring thaw periods may be represented by jump discontinuities at these times.

In general, both the rate of deterioration during the major part of the year and the abrupt deterioration associated with spring thaws may be expected to be functions of local factors as well as environmental and climatic condi-

tions. It is felt that the latter conditions are very influential in determining the spring thaw discontinuities and represent in this case an annual adversity due to the Illinois environment and climate. However, even for the same locality it must be noted that variations in spring thaws from year to year may be significant. Quantitative results for other localities and other times may be obtained through satellite test programs, and this study may provide guidance for the design of such programs. In such programs, it is to be expected that the magnitudes of both the rate of deterioration and the spring thaw deterioration discontinuities would be functions of both time and locality.

The data for approximately 134 in-test flexible pavement sections are analyzed in the manner described. Rates of deterioration and magnitudes of spring thaw deterioration are first determined as functions of a pavement thickness index. The thickness index employed was developed and utilized in HRB SR 61E and represents the structural composition of the pavement section for the case where mechanical properties of the component layers are considered constant. Next, relationships between the empirical coefficients determined in the preceding functions and the applied vehicle single-axle loads were found. For the case of the tandem-axle loads, single-axle equivalents were used. Finally, all the equations were combined to yield a general expression for PSI in terms of the local factors considered.

The present serviceability index is considered synonomous with the present serviceability rating and related to various other serviceability parameters, such as slope variance, rut depth, and roughness. These parameters are related in general descriptive analytic expressions and are found to be independent of pavement structural design, applied load, and number of applications. Such independence may be expected, inasmuch as all the indices involved depend only on subjective observations or objective measurements (which implicitly take into account structural composition) of the pavement surface. Using these relationships, one approach to modifying the mathematical expression for PSI presented in HRB SR 61E is suggested (though not necessarily recommended). The cracking-plus-patching index could not be included because insufficient data were available to establish any valid trend.

Another trend established in the study was a correlation between Benkelman beam creep speed deflections and local factors considered. Plotting deflections versus time for the two-year duration of the AASHO Road Test reveals significantly higher deflections in the spring and lower ones in the winter. This behavioral pattern was assumed to be described reasonably well by a sine wave of constant amplitude oscillating about a displaced horizontal base line. It is realized that the assumptions of constant amplitude and horizontal base line are subject to subsequent modification; however, for the two-cycle (two-year) duration of the AASHO Road Test no other realistic conclusion could be obtained. The amplitude and level of the base line are determined as functions of the thickness index and the applied single-axle load. The given relationship between time and number of applications allows the overall relationship for creep speed deflection to be expressed in terms of number of applications rather than time.

The behavior of slope variance and rut depth as a function of number of applications is qualitatively similar to the present serviceability index behavior and possibly might be handled in essentially the same manner. Roughness behavior seems to increase as number of applications increases, and exhibits no significant discontinuities during the spring thaw seasons. However, equations for these cases are not developed. The term "roughness" as used herein refers to measurements made with the Bureau of Public Roads roughometer.

Similar correlations between creep speed deflection and slope variance, rut depth, or roughness are attempted. Utilizing observations made in this study and equations developed in HRB SR 61E, a limited relation among creep speed deflection, rut depth, and number of load applications is given. A similar approach to the slope variance index seems feasible, although it is not reported herein. However, no relation between creep speed deflection and roughness seems apparent in the data studied.

An attempt to analyze the performance of overlaid sections was not successful. Although a few sections belonging to the higher design load category did indicate gradual deterioration with increasing number of load applications (similar to the constant rate of deterioration observed for the normal in-test sections which were not overlaid), the general trend of performance for overlaid sections indicates deterioration is negligible. Several inconsistencies in behavior were noted, and this fact made correlation attempts especially difficult.

It is felt that the trends and forms of equations developed herein, or some modification thereof, offer considerable promise toward the inclusion of a serviceability criterion in the current art of flexible pavement design. Examples have been presented to illustrate such an application. In addition, a knowledge of the design thicknesses and estimated traffic will permit an estimated pavement life to be predicted on the basis of these equations. The large amount of scatter encountered when comparing calculated and observed values suggests a cautious approach to the unqualified acceptance of values obtained from the developed equations; however, the many inconsistencies in observed data (even for replicate sections) account for much of this scatter and point out the extreme difficulties in maintaining adequate control on such large projects. Despite this scatter, it is felt the trends presented are valid and form a reasonable basis for other studies.

Perhaps the greatest limitation of this analysis is its restriction to the environmental conditions of the AASHO Road Test. The quantitative coefficients contained in the developed equations are for the specific conditions encountered in this test program and can not necessarily be extrapolated to other localities. It is only by means of a comprehensive satellite test program that quantitative estimates for these coefficients in other regions can be obtained. Another serious restriction of the current analysis is the exclusion of material properties from consideration. Although there are a few cases of treated base courses examined at the Road Test, these data are insufficient to develop any functional relationships and are not included. Thus, this analysis concerns itself with only one type of

material for each of the component layers—surface, base, subbase, and subgrade. It seems obvious that the effect of material properties needs to be examined by means of satellite test programs.

RIGID PLATE BEARING TESTS

Nondimensional techniques have been used to analyze data from rigid plates bearing on flexible pavement sections and various components of these sections. Response equations have been developed to describe pavement load-deflection behavior under a variety of different conditions. The data used were the results of the Hybla Valley test program and include effects of different test procedures, temperature, deflections at various layers, and limited repetitive loading (up to 75 repetitions).

It is noted that temperature decrease has an effect of increasing the "stiffness" of the soil-pavement system. Also, the load-deflection response of deflection-controlled and load-controlled tests is found to differ. This indicates that caution must be used when attempting any correlation with rigid plate bearing tests. The total deflections from original datum due to repetitive loading increase substantially (up to 80% or more for 75 load applications) compared to single-application deflections. The expression of surface, base course, and subgrade deflections by analytic equations permits an investigation of the effect of layer deflections. Surface deflection alone may be misleading in some cases unless the layer contributing the largest portion of the deflection is known. Although not specifically able to be deduced in this investigation due to its relatively short duration, it appears evident that rigid plate deflection data would exhibit a response similar to creep speed deflections, and correlations between the two could be obtained. Layer deflections take on added significance when it is noted that some studies have indicated no correlation between serviceability parameters and surface deflections. However, Walker et al. (27) have observed that crack frequency is related statistically to the subbase modulus and deduced that subbase deformations regardless of season contributed to the occurrence of surface cracks. They also concluded that deflection measurements of the individual layers are useful in showing the relationship between pavement cracking and the properties of each layer.

This study has illustrated a use of dimensional analysis in analyzing experimental data. Indications are that both model and prototype investigations designed and conducted on the basis of nondimensional techniques might prove effective in reducing unnecessary duplication and contribute toward the development of rational design criteria for flexible pavements.

LIMITATIONS

It must be noted that the results herein are restricted by the range of each local factor considered, soil type, and mechanical properties of the pavement components. Extrapolation of the trends established beyond the limitations imposed must be approached with caution. Furthermore, consideration of the wide scatter present in most cases and the behavioral inconsistencies noted (even for replicate sections) prohibit the establishment of anything more than behavioral trends. The developed equations are not to be regarded as quantitative design equations. Nevertheless, it is felt that the established trends are representative of behavioral patterns and may prove useful in subsequent analyses. Another important consideration is that the data lend themselves to a wide variety of different analytic expressions which seem to describe equally well the behavioral patterns over the relatively small range observed. Only one equation has been selected in each instance herein. It is always open to question whether the chosen equation is the "best" representation, especially when extrapolated beyond the range of variables studied. In this sense, the trends indicated herein represent first approximations and are subject to modifications with the advent of additional experimental results and the completion of various satellite programs related to the interrelations among parameters considered herein.

REFERENCES

1. Makamura, V. F., and Michael, H. L., "Serviceability Ratings of Highway Pavements." Purdue University, Reprint CE 197 (July 1963).
2. *The AASHO Road Test. Report 5. Pavement Research. HRB Spec. Report 61E* (1962).
3. Carey, W. H., Jr., and Irick, P. E., "The Pavement Serviceability-Performance Concept." *HRB Bull. 250*, pp. 40-58 (1960).
4. Benkelman, A. C., and Williams, S., "A Cooperative Study of Structural Design of Non-Rigid Pavements." *HRB Spec. Report 46* (1959).
5. Ingimarsson, G. R., "An Analysis of Hybla Valley Rigid Plate Bearing Data." *HRB Bull. 289*, pp. 22-41 (1961).
6. Kondner, R. L., and Krizek, R. J., "Non-Dimensional Techniques Applied to Rigid Plate Bearing Tests on Flexible Pavements." *HRB Bull. 289*, pp. 80-90 (1961).

7. HOUSEL, W. S., "Discussion—An Analysis of Hybla Valley Rigid Plate Bearing Data." *HRB Bull. 289*, pp. 41-43 (1961).
8. KRIZEK, R. J., and KONDNER, R. L., "Strength-Consistency Indices for a Cohesive Soil." HRB Record 48, pp. 1-18 (1964).
9. *Road Test One-MD. HRB Spec. Report 4* (1952).
10. *The WASHO Road Test. Part 1. Design, Construction, and Testing Procedures. HRB Spec. Report 18* (1954).
11. *The WASHO Road Test. Part 2. Test Data, Analyses, and Findings. HRB Spec. Report 22* (1955).
12. BENKELMAN, A. C., "Analysis of Flexible Pavement Deflection and Behavior Data." *HRB Bull. 210*, pp. 39-48 (1959).
13. *The AASHO Road Test. Report 1. History and Description of Project. HRB Spec. Report 61A* (1962).
14. *The AASHO Road Test. Report 2. Materials and Construction. HRB Spec. Report 61B* (1962).
15. *The AASHO Road Test. Report 3. Traffic Operations and Pavement Maintenance. Spec. Report 61C* (1962).
16. *The AASHO Road Test. Report 4. Bridge Research. HRB Spec. Report 61D* (1962).
17. *The AASHO Road Test. Report 6. Special Studies. HRB Spec. Report 61F* (1962).
18. HELMER, R. A., "How Oklahoma Uses Its Present Highways to Establish Criteria for Evaluating Future Structural Design Need." *HRB Bull. 210*, pp. 25-38 (1959).
19. ROBESON, F. A., "Accelerated Traffic Tests at Eglin Field, Florida." *Proc. HRB*, Vol. 24, pp. 55-70 (1944).
20. WOODS, K. B., and GREGG, L. E. "Pavement Performance Related to Soil Texture and Compaction." *Proc. HRB*, Vol. 44, pp. 426-440 (1944).
21. WILLIAMS, S., and LEE, A., "Load-Deflection Study of Selected High-Type Flexible Pavement in Maryland." *HRB Bull. 177*, pp. 1-20 (1958).
22. HICKS, L. D., "Flexible Pavement Deflection Study in North Carolina." *Proc. HRB*, Vol. 39, pp. 403-415 (1960).
23. QUINN, B. E., and DEVRIES, T. W., "Highway Characteristics As Related to Vehicle Performance. *HRB Bull. 250*, pp. 20-39 (1960).
24. DUNLAP, W. A., and STARK, L. E., "Deflection Tests on Texas Highways." *HRB Bull. 269*, pp. 52-65 (1960).
25. NICHOLS, F. P., JR., "Flexible Pavement Research in Virginia." *HRB Bull. 269*, pp. 35-51 (1960).
26. ZIMPFER, W. H., BRANSFORD, T. L., and GARTNER, W., JR., "A Tentative Flexible Pavement Design Method for Florida." *HRB Bull. 269*, pp. 16-34 (1960).
27. WALKER, R. D., YODER, E. J., SPENCER, W. T., and LOWRY, R., "Significance of Layer Deflection Measurements." *HRB Bull. 321*, pp. 63-81 (1962).
28. AHLBERG, H. L., and BARENBERG, E. J., "The University of Illinois Pavement Test Track—A Tool for Evaluating Highway Pavements." *HRB Record 13*, pp. 1-21 (1963).
29. PAINTER, L. J., "Analysis of AASHO Road Test Asphalt Pavement Data by the Asphalt Institute." *HRB Record 71*, pp. 15-38 (1965).
30. SEBASTYAN, C. Y., "The Benkleman Beam Deflections as a Measure of Pavement Strength." *Proc. Canadian Good Roads Assn.* (1960).
31. KONDNER, R. L., and KRIZEK, R. J., "Correlation of Load Bearing Tests on Soils." *Proc. HRB*, Vol. 41, pp. 557-590 (1962).
32. KONDNER, R. L., "A Non-Dimensional Approach to the Vibratory Cutting, Compaction and Penetration of Soils." *Tech. Report 8*, by the Johns Hopkins Univ. to the U.S. Army, Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss. (1960).
33. YODER, E. J., "Flexible Pavement Deflections—Methods of Analysis and Interpretation." *Proc. Assn. of Asphalt Paving Technol.*, Vol. 31, pp. 260-288 (1962).
34. BURMISTER, D. M., "The Theory of Stresses and Displacements in Layered Systems and Applications to Design of Airport Runways." *Proc. HRB*, Vol. 23, pp. 126-149 (1943).
35. BURMISTER, D. M., "Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System: Influence Diagrams and Practical Applications." *Proc. HRB*, Vol. 35, pp. 773-814 (1956).
36. FOX, L., "Computation of Traffic Stresses in a Simple Road Structure." *Road Research*, Tech. Paper No. 9, Road Research Lab., London, England (1948).
37. JONES, A., "Tables of Stresses in Three-Layer Elastic Systems." *HRB Bull. 342*, pp. 176-214 (1962).
38. PALMER, L. A., and BARBER, E. S., "Soil Displacement Under a Circular Loaded Area." *Proc. HRB*, Vol. 20, pp. 279-286 (1940).
39. BAKER, R. F., and PAPAZIAN, H. S., "The Effect of Stiffness Ratio on Pavement Stress Analysis." *Proc. HRB*, Vol. 39, pp. 61-85 (1960).
40. HOUSEL, W. S., "A Practical Method for the Selection of Foundations Based on Fundamental Research in Soil Mechanics." *Eng. Res. Bull. No. 13*, Univ. of Michigan (1929).
41. KONDNER, R. L., and KRIZEK, R. J., "A Non-Dimensional Approach to the Static and Vibratory Loading of Footings." *HRB Bull. 277*, pp. 37-60 (1960).
42. KONDNER, R. L., "Hyperbolic Stress-Strain Response: Cohesive Soils." *Proc. ASCE, Jour. Soil Mech. and Fdns. Div.*, Vol. 89, No. SM1, pp. 115-143 (1963).
43. KONDNER, R. L., "Friction Pile Groups in Cohesive Soil." *Proc. ASCE, Jour. Soil Mech. and Fdns. Div.*, Vol. 88, No. SM3, pp. 117-149 (1962).

APPENDIX A

PERFORMANCE STUDIES

TEST PROGRAMS REVIEWED

General Description

The primary goal of any performance study is to determine the effect of controlled test traffic on pavement behavior. With the gradual traffic growth on highways, it has been necessary to obtain factual data concerning the effect of the magnitude and number of applications of a given type axle load on pavement sections of various design thicknesses. To achieve this end, a series of large-scale highway research projects have taken place; namely, the One—MD, WASHO, and AASHO Road Tests. The first two projects were planned on a regional basis and involved only a few variables, whereas the third was conceived on a national basis and had involved in its design more variables in terms of load, pavement thicknesses, and other deterioration factors. In addition, the AASHO Road Test was conducted for a longer period of time in order to attain a larger number (more than one million) of controlled load applications and to take into account effects of climatic conditions. Besides the larger projects mentioned, there have been a number of similar smaller-scale studies conducted, generally at the level of a state highway department or university. Although several studies are reviewed herein, only data from the AASHO Road Test are utilized in the subsequent analysis.

Road Test One—MD

Road Test One—MD, which was the first of a series of large highway research projects, was conducted on a two-lane portland cement concrete pavement. The principal object of the test was to determine the relative effects on a particular concrete pavement of four different axle loadings on two vehicle types. The loads employed were 18,000 and 22,400 lb on single axles and 32,000 and 44,800 lb on tandem axles. The tests were conducted on a 1.1-mile section of rigid pavement constructed in 1941 on US 301 approximately 9 miles south of La Plata, Md. The pavement consisted of two 12-ft lanes, each having a 9-7-9-in. cross-section reinforced with wire mesh. The four separate test sections were founded on soils of various classifications. Because the controlled-traffic tests were conducted from June through December 1950, the pavement sections actually had a pretest life of approximately ten years. Upon completion of the test traffic phase, the supplemental strain and deflection testing program was delayed due to frozen subgrades during January and February 1951. This, in turn, delayed the extensive soil sampling and testing program. Data gathering was completed in April 1952, and ultimately included approximately 6,000 soil tests and 12,000 strain and deflection readings. In terms of climate and soil conditions, the site was representative of many eastern and midwestern states. The details of this road test, together with its major findings, are contained in HRB SR 4 (*9*).

WASHO Road Test

The WASHO Road Test was a regional research project sponsored by the Western Association of State Highway Officials and closely followed Road Test One—MD. Two test tracks of flexible pavement sections were constructed at a site located about 11 miles south of Malad, Idaho. The soils on the site were silt loams bordering on silty clay loams classified as A-4(8). The flexible pavement sections consisted of various combinations of 0-, 4-, 8-, 12-, and 16-in. subbase thicknesses covered with either 2 in. of base course and 4 in. of surface course or 4 in. of base course and 2 in. of surface course. Each roadway consisted of two 12-ft lanes with 6-ft shoulders. The axle weights of the test vehicles were 18,000 and 22,400 lb for single-axle units and 32,000 and 40,000 lb for tandem-axle units. After completion of preliminary testing and installation of instruments, regular test traffic operation began in November 1952 and continued until May 1954, except for one spring and two winter periods. Although the weather was less severe than normal for the area, it was considered reasonably representative of large areas of the west. Further details on the design, construction, and test procedures are given in HRB SR 18 (*10*), and test data, analyses, and findings are contained in HRB SR 22 (*11*).

In the treatment and presentation of test data, standard statistical methods were employed. Some of the principal findings of this test program centered around the superior behavior of the pavement sections with 4 in. of surface course and the relative behavior of inner and outer wheel paths. The results of one analysis of these data have been presented by Benkelman (*12*). This project, together with Road Test One—MD, emphasized the need for a larger and more comprehensive test program including a greater number of variables and significantly more load applications. This need was fulfilled to a large degree by the AASHO Road Test.

AASHO Road Test

In February 1955 the Highway Research Board accepted the responsibility of administering and directing the AASHO Road Test. The project was located near Ottawa, Ill., about 80 miles southwest of Chicago. This site was chosen because the soil within the area was uniform and of a type representative of that found in large areas of the country, because the climate was typical of that found

throughout much of the northern United States, and because much of the earthwork and pavement construction could ultimately be utilized in the construction of a section of the National System of Interstate and Defense Highways.

The test facilities consisted of two small loops (numbered 1 and 2) and four larger loops (numbered 3 through 6). Test bridges were constructed at four locations in two of the larger loops. Each loop was a segment of a four-lane divided highway whose parallel roadways or tangents were connected by a turnaround at each end. In Loop 1 the tangent lengths were 2,000 ft; in Loop 2, 4,400 ft; and in Loops 3 through 6, 6,800 ft. No traffic operated over Loop 1 and all vehicles assigned to any one traffic lane of Loops 2 through 6 had the same axle arrangement-axle load combinations. In all loops the north tangents consisted of flexible pavement sections and the south tangents of rigid pavement sections. Each tangent was constructed as a succession of pavement sections called structural sections. As a rule, pavement designs varied from section to section, the minimum length of a section being 100 ft in Loops 2 through 6 and 15 ft in Loop 1. Sections were separated by short transition pavements and each structural section was separated into two pavement test sections by the center line of the pavement.

Various combinations of each of the following thicknesses were included in the test program: surface course—1, 2, 3, 4, 5 and 6 in.; base course—0, 3, 6, and 9 in.; subbase course—0, 4, 8, 12 and 16 in. The exact combination for any given test section may be found in Table 2 of HRB SR 61E (*2*). In several instances replicate sections (two test sections having the same pavement design and load assignment) were studied. In the interest of uniformity, the soil making up the top 3 ft of embankment directly beneath the test pavements was taken from borrow areas near the project and may be classified generally as an A-6 or A-7-6 soil.

The axle loads and arrangement applied to each lane of each loop are given in Table A-1. Controlled test traffic began in November 1958 and continued for 25 months until December 1960. The number of vehicles operating in any given lane was selected so that axle load applications could be accumulated at the same rate in each of the ten traffic lanes. During this period a total accumulation of 1,114,000 axle load applications was attained. Whenever possible, traffic was operated at 35 mph on the test tangents.

The primary objectives of the AASHO Road Test were concerned with the performance of the test sections as constructed. Consequently, maintenance operations were held to a minimum in any section that was still considered under study. When the serviceability of any given section dropped to a specified level, the section was considered to be out of test and maintenance or reconstruction was performed as needed.

Although the AASHO Road Test included numerous auxiliary studies, such as the effectiveness of asphaltic concrete overlays, special base-type experiments (crushed stone, gravel, cement-treated and bituminous-treated gravel), and effectiveness of paved shoulders, the first objective of the project was to obtain relationships between the performance of the pavement and the pavement design variables for various loads. It is this latter objective to which the remainder of this section is primarily devoted. Fairly elaborate details of all phases of the AASHO Road Test may be found in the reports on the project (*13, 14, 15, 16, 2, 17*).

TABLE A-1

AXLE LOADS AND ARRANGEMENT ON AASHO ROAD TEST PAVEMENTS

LOOP	LANE	AXLE LOAD AND ARRANGEMENT[a]
1	1	None
	2	None
2	1	2,000 lb SA
	2	6,000 lb SA
3	1	12,000 lb SA
	2	24,000 lb TA
4	1	18,000 lb SA
	2	32,000 lb TA
5	1	22,400 lb SA
	2	40,000 lb TA
6	1	30,000 lb SA
	2	48,000 lb TA

[a] SA = single axle; TA = tandem axle

Other Studies

In an effort to present a more complete picture of performance studies conducted on highway pavements, a few additional smaller-scale investigations are reviewed in the following. No doubt there are other studies which will not be covered, but those given here are considered the most pertinent.

Helmer (*18*) exploits the fact that to some extent all highways that have been built to a rational design can be used as research projects. Although records of design and construction may not have been kept in as much detail as is customary on a research project, the many miles of projects and the wide variety of conditions and environment available for study may often more than compensate for the lack of preliminary data. In his work Helmer studies the performance of 321 miles of pavement in Oklahoma and suggests how an analysis of these data may lead to a better understanding of problems of highway design.

Robeson (*19*) reports on several accelerated traffic test studies conducted on a test track at Eglin Field, Fla. The track included sections of an existing airfield pavement and large earth-moving vehicles were used to simulate airplane traffic. Separate test lanes for wheel loads of 15,000, 37,000 and 50,000 lb were provided and each wheel load was continued until 3,500 coverages of a strip three times the tire contact width had been made.

Woods and Gregg (*20*) report on the correlation of pavement performance with soil texture. Their observations stress the importance of adequate compaction and more extensive use of granular material for embankments, subgrades, and bases. Performance surveys of flexible pavements made during adverse weather conditions indicate wide variation in performance which can be directly attributed to the variations in soil textures and differential pavement thicknesses.

Williams and Lee (*21*) report on a cooperative program of load-deflection tests on several high-type flexible pavements in Maryland. Tests were made in the spring and fall at approximately 1,000 marked locations over a distance of about 85 miles on pavements in excellent condition and ranging in age from two to eleven years. It was found that the magnitude of the deflections of a certain pavement of constant design may vary to a great degree from site to site, and even at different points at the same site. Also, despite the comparatively large deflections at some test sites for the spring test series, there was no evidence of structural distress at these sites. An attempt to correlate the magnitude of the pavement deflection with the results of the subgrade soil surveys made prior to construction did not show any definite trends. In general, outer wheel path deflections were larger than those for the inner wheel path, especially for two-lane highways. The effect of changes in climatic conditions on pavement deflection was found to be quite marked. Inasmuch as the observed pavements were generally in excellent condition, it was not possible to correlate structural adequacy and total deflection.

Using deflections measured by means of a Benkelman beam, Hicks (*22*) surveyed flexible pavement structures in North Carolina over a period of three years and reports on a correlation of these deflections with observed pavement condition. Survey data on some 24 roads are given to represent various subgrade soils, base types, and bituminous surface course thicknesses. His conclusion is that a deflection survey is an excellent means of flexible pavement evaluation.

Quinn and DeVries (*23*) utilize the concept that the performance of a vehicle on a highway is dependent on the characteristics of both. In their work "performance" pertains to the vertical dynamic force that the wheels of a moving vehicle exert on the highway as a result of variations in the pavement profile. The effect of vehicle speed is seen to be significant in the special situation where a section of highway considered "smooth" produces a higher mean squared force than a section considered "rough."

Dunlap and Stark (*24*) present some of the experiences and results obtained from an investigation of pavement deflections in Texas. Benkelman beam deflections on 117 miles of flexible pavements lead to the observation that in many cases initial beam readings were biased due to temperature of the pavement surface, length of the deflection "trough," friction in the Benkelman beam and Helmer recorder, and field techniques in the operation of the Benkelman beam. Means are given for correcting the deflections when a recorder attachment is used on the Benkelman beam. Attempts to correlate the deflected length with pavement behavior were unsuccessful.

Nichols (*25*) describes the findings from the first experimental project of a program of experimental construction planned in Virginia. This study compares the performance of different pavement designs built at the same time on the same project and subjected to the same traffic. Four thicknesses (4, 5, 7 and 9 in.) of asphaltic concrete were incorporated in designs of the same total thickness. Some of the findings are that deflections and performance seem more closely allied with compaction than with pavement design characteristics, deflections are somewhat less abrupt in the sections with greater thickness of asphaltic concrete, rutting in the wheel tracks occurred to some extent throughout the project regardless of pavement design, and the deepest rutting occurred where the asphaltic concrete was the thickest.

Zimpfer, Bransford, and Gartner (*26*) utilize observed pavement performance in Florida to assist in developing a tentative flexible pavement design method. Representative pavements in rural locations with 10-year performance records and observed condition ratings of poor to very good were selected and actual field conditions were determined at each test site. Using quantitative measures of longitudinal profile, depth of rutting, and degree of surface cracking, a service rating was developed and incorporated into design requirements along with the normal design variables.

Walker et al. (*27*) investigated a series of Benkelman beam deflection measurements made on the US 31 test road near Columbus, Ind., for the purpose of determining the significance of layer deflection and in particular to ascertain whether correlations could be established between total deflection and pavement condition and between layer deflection and pavement condition. Comparisons were made among surface rutting, cracking, surface deflection, and layer deflections. Their observations were that (a) cracking was not shown to be related to subgrade type, bituminous pavement thickness, total deflection, or rutting; (b) more cracking occurred in the traffic lane than in the passing lane (probably due to heavier and greater traffic volume on the traffic lane); (c) subgrade type, lane, and pavement thickness affected the deflection data, whereas only subgrade type and lane affected the rutting data; and (d) a direct relationship existed between rutting and total deflection.

Ahlberg and Barenberg (*28*) reported on the development of a pavement test track at the University of Illinois. It was designed to apply simulated traffic loads at a high frequency and thereby investigate pavement performance, behavior, and failure modes under controlled conditions. Considerable emphasis is placed on the use of performance-serviceability concepts to evaluate the rate of pavement deterioration as a failure criterion.

Makamura and Michael (*1*) studied 60 pavement sections located within a 40-mile radius of Lafayette, Ind. The pavement sections varied in length from 0.50 to 12.75 miles (averaging about 5 miles) and totaled approximately 300 miles. Nineteen of the sections were rigid pavements, 22 were rigid with bituminous overlay, and 19 were flexible pavements. All types of pavement condition were included in each surface type. Their conclusions are that a rating panel method of evaluating pavement serviceability is practical, whereas the roughometer method is objective, simple, and accurate for rigid pavements but not a good predictor for overlay and flexible pavements.

SERVICEABILITY RATING

In keeping with the principal objective of the AASHO Road Test, an adequate and unambiguous definition of pavement performance was required. To provide dimen-

sions for the term "performance," a system was devised that is rational and free from the likelihood of bias due to the strong personal opinions of groups or individuals. The derivation of the pavement serviceability-performance system and the fundamental assumptions on which it is based have been given by Carey and Irick (*3*) and are not repeated here. It will suffice to present briefly the mechanics of its operation.

Present serviceability may be defined as the ability of a specific section of pavement to serve high-speed, high-volume mixed traffic in its existing condition. An individual present serviceability rating is an independent rating between the limits 0 (very poor) and 5 (very good) by an individual of the present serviceability of a specific section of roadway. All features not related to the pavement itself, such as right-of-way width, grade, alignment, shoulder and ditch condition, etc., are to be excluded from consideration. The present serviceability rating (PSR) is the mean of the individual ratings made by the members of a specific panel intended to represent all highway users. The present serviceability index (PSI) is a mathematical combination of values obtained from certain physical measurements of a large number of pavements so formulated as to predict the PSR for those pavements within prescribed limits. The performance index (PI) is a summary of PSI values over a period of time.

For the case of flexible pavements the physical measurements which were mathematically combined to form the PSI include slope variance, *SV*, rut depth, *RD*, and cracking and patching, $C + P$. A brief description of each of these terms (plus roughness, *R*, and Benkelman beam deflection, δ, with their respective dimensions is given in Table A-2. After linearizing, transformations were selected for each of these parameters; evaluation of the results from 74 flexible pavement sections (20 from AASHO Road Test, 4 off-site sections, 10 in Illinois, 20 in Minnesota, and 20 in Indiana) by the method of least squares yielded the relationship

$$\mathrm{PSI} = 5.03 - 1.91 \log (1 + \overline{SV}) - 1.38\, \overline{RD}^2 - 0.01\, (C + P)^{\frac{1}{2}} \qquad \text{(A-1)}$$

There are many other possibilities for this equation which may take into account different linearizing functions and additional variables. Although there is nothing to suggest that the equation is in its optimum form, it is nonetheless the form employed by the AASHO Road Test staff.

Such a relation as Eq. A-1 is especially necessary for the AASHO Road Test due to the length of the pavement sections. Most highway users consider a high-speed ride over a pavement necessary before they will rate it, and since the Road Test sections were too short (as short as 100 ft) to provide a subjective evaluation of their ability to serve traffic, objective measurements conducted on the short sections had to be selected and utilized as performance indices in such a way that short pavements could be evaluated as if they were much longer. Based on the 74 sections evaluated, PSI values as computed by Eq. A-1 differed from PSR values by less than 16 percent for flexible pavements. Hence, PSI may be treated essentially as PSR whenever convenient.

TABLE A-2

DESCRIPTIONS AND DIMENSIONS OF VARIOUS SERVICEABILITY INDICES USED ON AASHO ROAD TEST

INDEX	DIMENSION	DESCRIPTION
PSI	———	Mathematically computed index designed to correlate with subjective panel rating.
SV	———	Variance of longitudinal slope as measured by a profilometer.
RD	in.	Maximum vertical displacement of a point on the pavement surface as measured from the center of a 4-ft transverse straight edge.
R	in./mile	Roughness of pavement as measured by the Bureau of Public Roads roughometer.
C	sq ft/1,000 sq ft	Area of pavement surface exhibiting class 2 or class 3 cracking.[a]
P	sq ft/1,000 sq ft	Repair of the pavement surface by skin patching or deep patching.
δ	in.	Benkelman beam creep speed deflection recorded when the load approaches and leaves the probe point at a speed of 2-3 mph.

[a] 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

DATA ANALYZED

All of the data analyzed in the remainder of this appendix were obtained from the AASHO Road Test. Furthermore, the majority of these data are contained in Data Systems 5121-F and 7322-D. Data System 5121-F contains routine Benkelman beam deflections on the flexible pavement sections of all loops; Data System 7322-D includes serviceability index data and associated measurements for all test road sections (although only flexible pavement data were utilized). These data systems (together with most other systems) are described in more detail in Appendix I of HRB SR 61E (*2*).

Inasmuch as the AASHO Road Test was primarily designed to provide information relative to pavement structure, certain aspects of normal pavement serviceability (condition of shoulders, surface friction, etc.) were not considered. The performance indices which have been considered are present serviceability index, PSI, Benkelman beam deflections, δ, slope variance, *SV*, rut depth, *RD*, roughness, *R*, and cracking plus patching, $C + P$. The structural variables taken into account are the thicknesses of the surface course, *a*, base course, *b*, and subbase course *c*. In addition, the vehicle axle load, *P*, and number of applications, *N*, are considered. In general, the data reported in these data systems were taken at two-week intervals. In some cases correlation between the indices taken from each of the two data systems for supposedly the same day may actually vary by four or five days. However, it is reasonable to assume that the respective indices would not be subject to excessive variation during this time.

The major effort is expended to establish relationships among the quantities previously mentioned for the pave-

ment sections which did not fail during the test program. In the design of the overall AASHO Road Test failures were definitely contemplated and certain pavement sections were knowingly underdesigned to accomplish this. Similarly, some sections were overdesigned in order to assure a complete span of the performance range. Many of the sections reached a stage where maintenance was required in order to facilitate continued traffic operation. The maintenance operations consisted of either minor repairs, overlays, or complete reconstruction. Except for specific comments regarding the overlaid sections, the analysis herein is confined to those data obtained from flexible pavement sections up to the time the section failed or was overlaid. Of course, if neither occurred the data were accumulated throughout the duration of the project.

Within practical limitations, the material properties of the subgrade and each layer of pavement thickness were maintained constant in the construction process. It is certain that variations in these properties must have occurred throughout the duration of the project. However, because one of the most important variables considered in this research was the number of applications of a given load, every effort was made to maintain traffic operations. Consequently, few organized data regarding material property variations were obtained from the actual pavement sections analyzed herein because the gathering of such data would have delayed the accumulation of load applications. Two exceptions to this are the Loop 1 pavement sections and the failed pavement sections in Loops 2 through 6. Loop 1 was a non-traffic loop constructed primarily for the purpose of performing various tests and making comparisons with results of similar pavement sections in the traffic loops. When pavement sections in the traffic loops failed, very often material property tests were made on the various components (subgrade, base course, etc.) Thus, although limited amounts of material property data are available, whether or not such data from non-traffic pavement sections or failed pavement sections are representative of the actual conditions existing in the in-test pavement sections is open to doubt. This study does not consider explicitly such material variations as changes in moisture content or density, but considers them only insofar as they contribute to the overall deterioration of the pavement.

In other sections certain relationships between various performance indices are investigated. Correlations are made between PSI and another index, such as *SV*, *RD*, and *R*. It is realized that the PSI utilized has been calculated by use of Eq. 46 and already includes an assigned dependence on each of these indices. However, PSR (as previously defined) is independent of any direct relation to these other indices and furthermore is essentially equivalent in magnitude to PSI. In this sense, PSI and PSR may be used interchangeably and regarded as independent of δ, *SV*, *RD*, *R*, and $C + P$. Such an approach is adopted in the analysis of these data and various relationships are studied. These relationships may possibly suggest modifications to Eq. A-1.

The AASHO Road Test data contain performance-serviceability records for approximately 288 flexible pavement sections in Loops 2 through 6. Approximately 154 of these sections failed (PSI < 1.5) early during the first spring thaw, $N < 140{,}000$, and were overlaid to carry the remaining traffic. Of these 154 sections, the performance of 44 was observed for the entire period of traffic operation, the other 110 were considered out-of-test and no further serviceability records were maintained. Performance-serviceability data were plotted for a total of 178 sections—134 which survived the first spring thaw and 44 for which records were maintained. Of the remaining 134 sections,

TABLE A-3

DISTRIBUTION OF NUMBER OF AASHO ROAD TEST SECTIONS USED IN THIS ANALYSIS

			NUMBER OF SECTIONS				
LOOP	LANE	LOAD[a] (KIPS)	FAILED AFTER FIRST SPRING THAW, $N < 140{,}000$	OVERLAID AFTER FIRST SPRING THAW AND PERFORMANCE OBSERVED	FAILED DURING SECOND SPRING THAW, $N < 610{,}000$	OVERLAID AFTER SECOND SPRING THAW AND PERFORMANCE OBSERVED	IN TEST THROUGH ENTIRE TEST DURATION, $N = 1{,}114{,}000$
2	1	2.0 SA	2	–	3	–	17
	2	6.0 SA	10	1	5	3	7
3	1	12.0 SA	24	3	3	2	3
	2	24.0 TA	25	1	4	4	1
4	1	18.0 SA	17	5	6	5	7
	2	32.0 TA	16	6	9	4	5
5	1	22.4 SA	19	10	6	3	5
	2	40.0 TA	17	6	8	6	5
6	1	30.0 SA	13	9	10	5	7
	2	48.0 TA	7	3	15	9	8

[a] SA = single axle; TA = tandem axle.

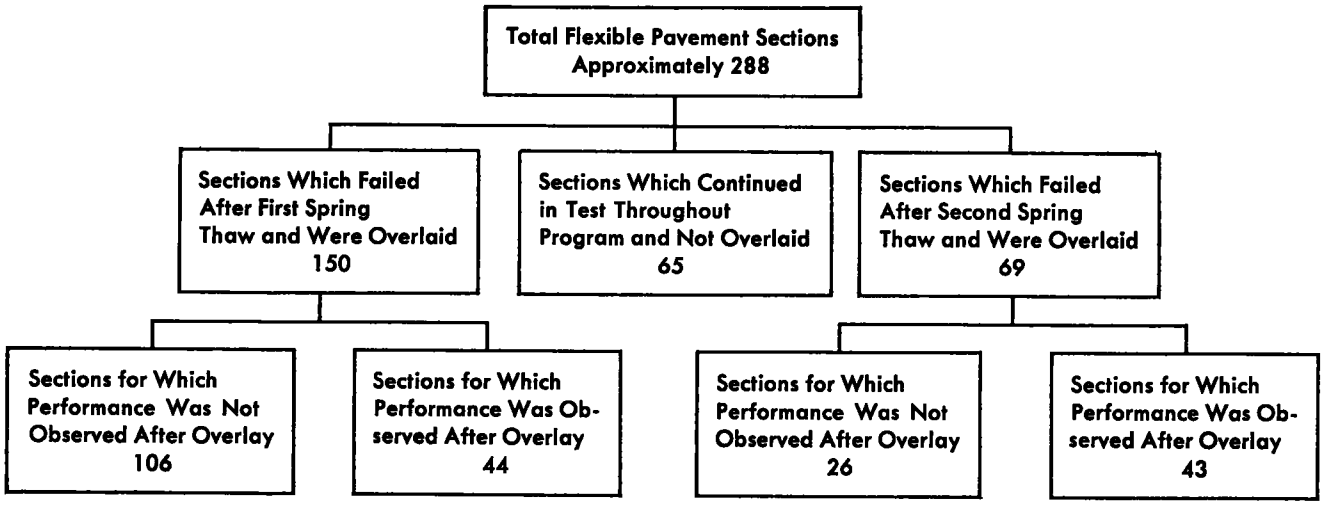

Figure A-1. Flow chart for sections analyzed at AASHO Road Test.

65 survived the total number of load applications. These sections were purposely overdesigned and showed low rates of deterioration and losses in serviceability during the spring thaws. The remaining 69 sections failed during the second spring thaw and were overlaid to permit the continuance of test traffic as on the in-test sections. However, the performance of only 43 such sections was observed until the end of the test; serviceability records for the remaining 30 sections were discontinued. Table A-3 and Figure A-1 show the general breakdown of the number of AASHO Road Test sections used in this analysis. In addition, Table A-4 gives a more detailed description of the specific sections employed in the development of each equation.

METHOD OF ANALYSIS

General Discussion

As previously mentioned, the many variables which affect pavement performance make it extremely difficult to effectively isolate any one variable for study. To objectively evaluate the influence of any single variable on pavement performance, all other pertinent variables would have to be maintained constant. This becomes virtually impossible in any large-scale program such as the AASHO Road Test, particularly in view of the "intangible" variations in construction procedure and climatic conditions. Hence, even a cursory investigation of available data reveals that it is extremely difficult to realistically and objectively deduce any "exact" equation relating the variables under consideration, not only theoretically but also empirically. Nonetheless, a little effort unveils the presence of several trends among the variables; it is the purpose herein to present an analysis of several of these trends. It must be emphasized that only trends are established and not exact relationships—despite the development of analytic expressions to approximate these trends.

For the most part, performance data considered henceforth are those taken for the inner wheelpath. Occasionally outer wheelpath data have been examined to investigate any divergence. The superior performance of the inner wheelpath over the outer wheelpath has been reported in the WASHO test results and other studies. With regard to the AASHO Road Test, it has been pointed out elsewhere (*2*) that the performance equations for the outer wheelpath had a better correlation with the performance equations of the entire section. It seems reasonable to expect, however, that similar patterns will result for both wheelpaths. Hence, the more difficult inner wheelpath correlation is presented in this study.

TABLE A-4

SECTIONS USED IN DEVELOPMENT OF EACH EQUATION IN THIS ANALYSIS

VARIABLES	EQUATION	FIGURE	SECTIONS USED
PSI, *a, b, c, P, N*	10	2, 3	All 134
PSI, *SV*	15	4	109, 122, 155, 256, 264, 268, 303, 455, 470, 479, 577, 582, 623, 631, 731, 746
PSI, *RD*	16	5	109, 122, 139, 140, 155, 156, 256, 303, 309, 446, 455, 478, 577, 601, 623, 626, 746, 755, 776
PSI, *R*	17	6	122, 139, 140, 155, 256, 303, 309, 441, 455, 478, 577, 601, 623, 626, 746, 755, 776
δ, *a, b, c,* time	27	8, 9	65 in lane 1 of Loops 2, 3, 4, 5, 6 and lane 2 of Loop 2
δ, *RD, N*	29	10	109, 139, 155, 333, 455, 581, 623, 776, and data from Fig. 95, HRB SR 61E (*2*)

Analysis of PSI-Local Factor Relationship

The first task is to develop a relationship between the PSI and the various local factors which influence it, such as surface thickness, *a*, base thickness, *b*, subbase thickness, *c*, vehicle axle load, *P*, and number of applications, *N*. Such an approach intuitively assumes the constancy of all other variables, such as material properties, etc.

As an initial step toward obtaining the desired relationship, PSI may be plotted versus *N* for any given section wherein *a*, *b*, *c*, and *P* are constant. One example of such a graph is Figure A-2, for two replicate test sections in lane 2 of Loop 6. The performance of the two sections is quite similar, according to expectations. Unfortunately, however, such is not the case for all replicate test sections. In Figure A-3 the plots nearly parallel each other, but have an entirely different origin, which probably indicates a variation in construction procedure. Figure A-4 shows two replicate sections which have virtually the same PSI at the start of testing but behave in entirely different manners during the test program. Figure A-5 shows two replicate sections which have nearly the same starting point; however, the spring thaws seem to have manifested a greater effect on one section and failure occurred after the second spring thaw (when it was overlaid), whereas its companion section continued in test throughout the project. Finally, Figure A-6 indicates two replicate sections beginning at the same point but demonstrating significantly different behavior thereafter; one section was overlaid after the first spring thaw, whereas the other received the same treatment after the second spring thaw. Obviously, it is virtually impossible to reconcile the varied performance of such replicate sections without additional investigation of other factors. Furthermore, such observed behavior for replicate sections casts suspicion on the results obtained from single test sections, and their unqualified acceptance can not be recommended. Hence, again the argument for a trend, and not an exact relationship, must be forwarded.

Referring to Figure A-2 (as well as most of the other 178 similar graphs), it is observed that these data can be approximated by a step function type curve as shown in Figure A-7. A similar approach involving log *N* was employed in a statistical analysis reported by Painter (29). Such an empirical fit assumes that the rate of deterioration, *S*, is constant during the majority of the year and from year to year; furthermore, the greatest deterioration (or loss in serviceability) occurs during a relatively short period of time (approximated by a jump discontinuity) immediately following the spring thaw. For many of the test sections the change in serviceability during the spring thaws is very small or negligible; this is particularly true for the overdesigned sections. It is conceivable that in a climate exhibiting no spring thaws the rate of deterioration would be a constant throughout the year. In general, an equation relating these two parameters (PSI and *N*) may be written

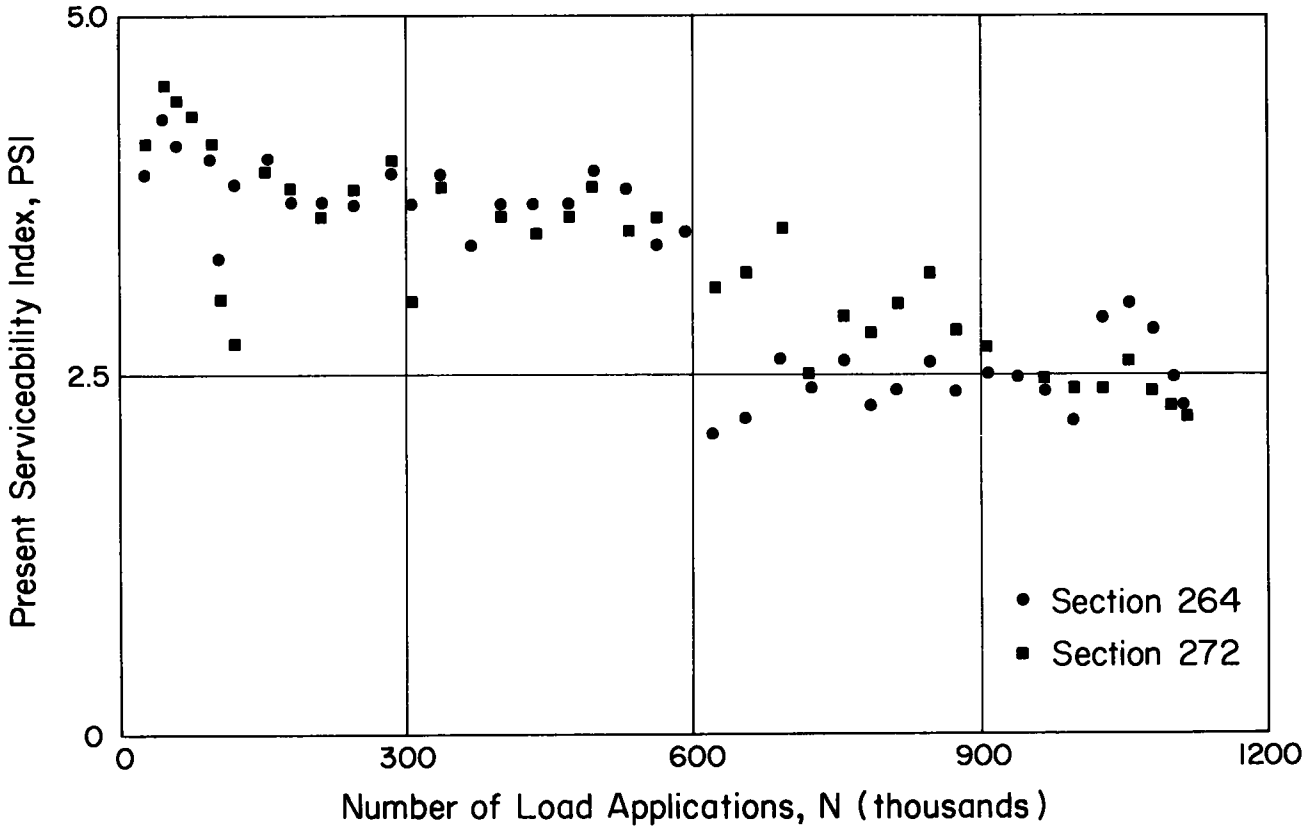

Figure A-2. Present serviceability index vs number of load applications; duplicability, sections 264 and 272.

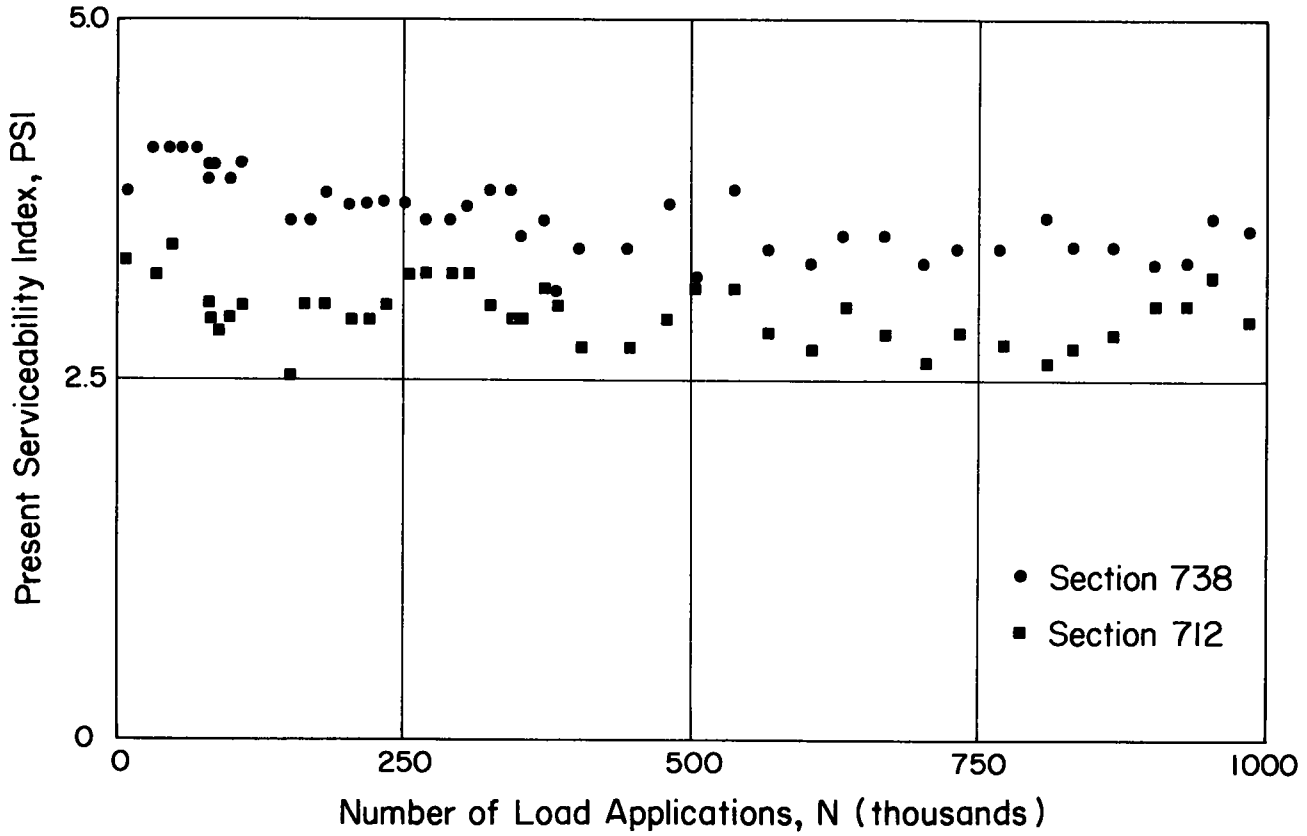

Figure A-3. Present serviceability index vs number of load applications; duplicability, sections 738 and 712.

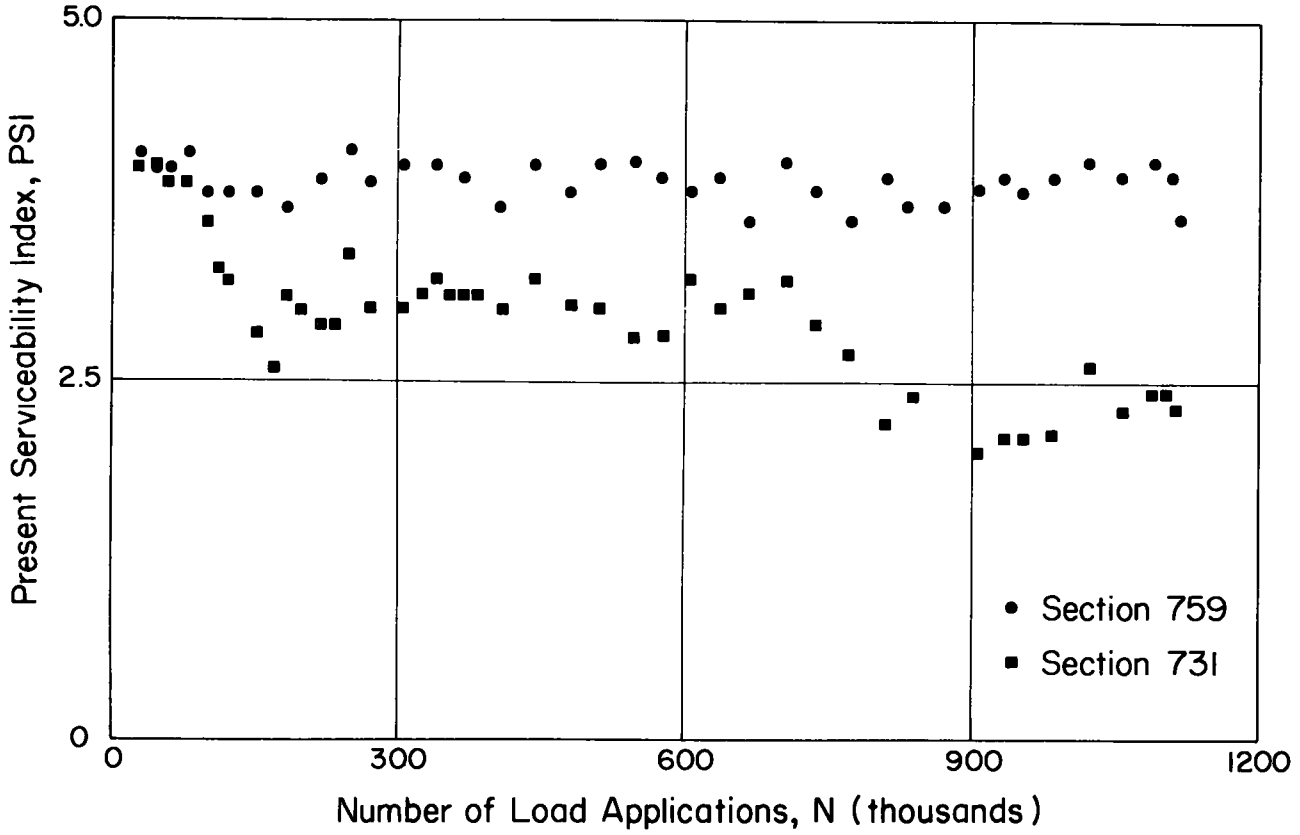

Figure A-4. Present serviceability index vs number of load applications; duplicability, sections 759 and 731.

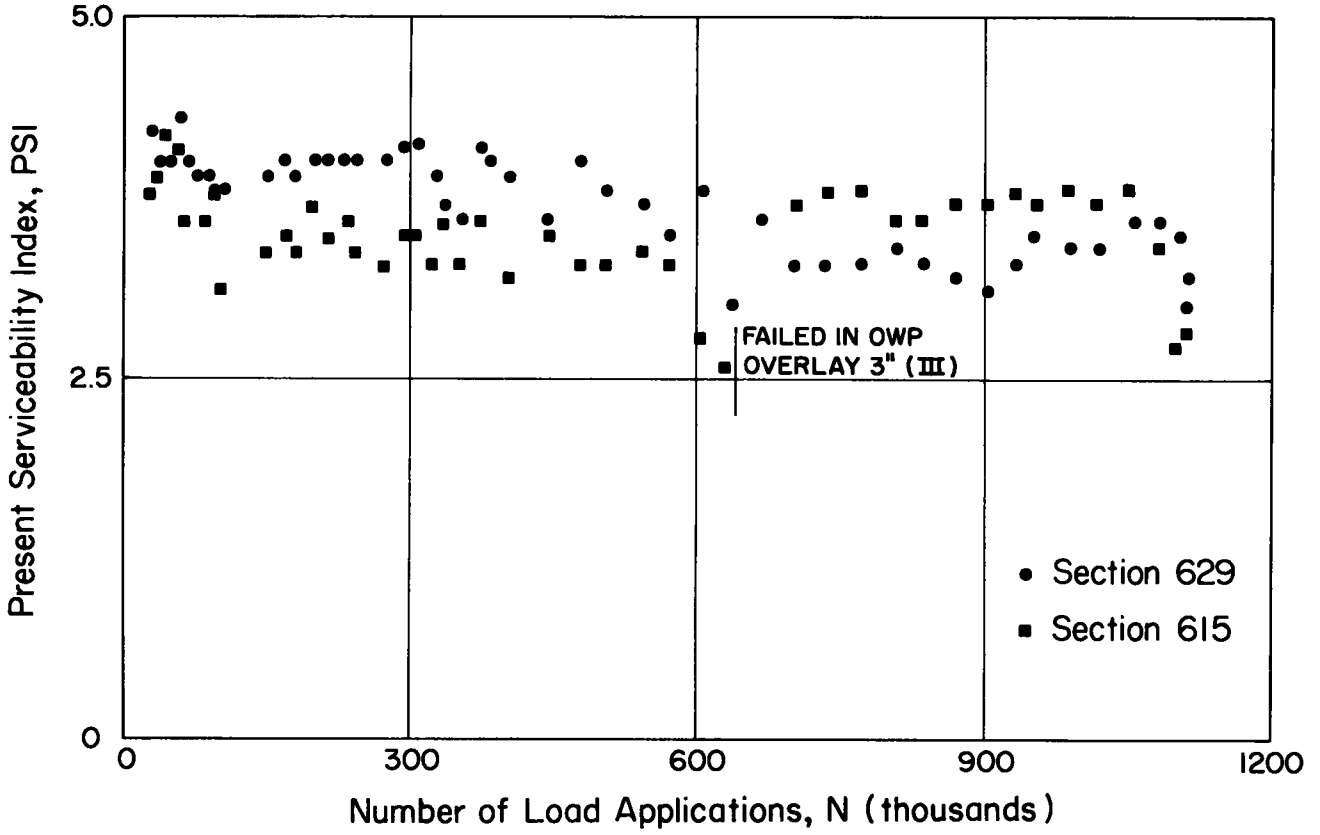

Figure A-5. Present serviceability index vs number of load applications; duplicability, sections 629 and 615.

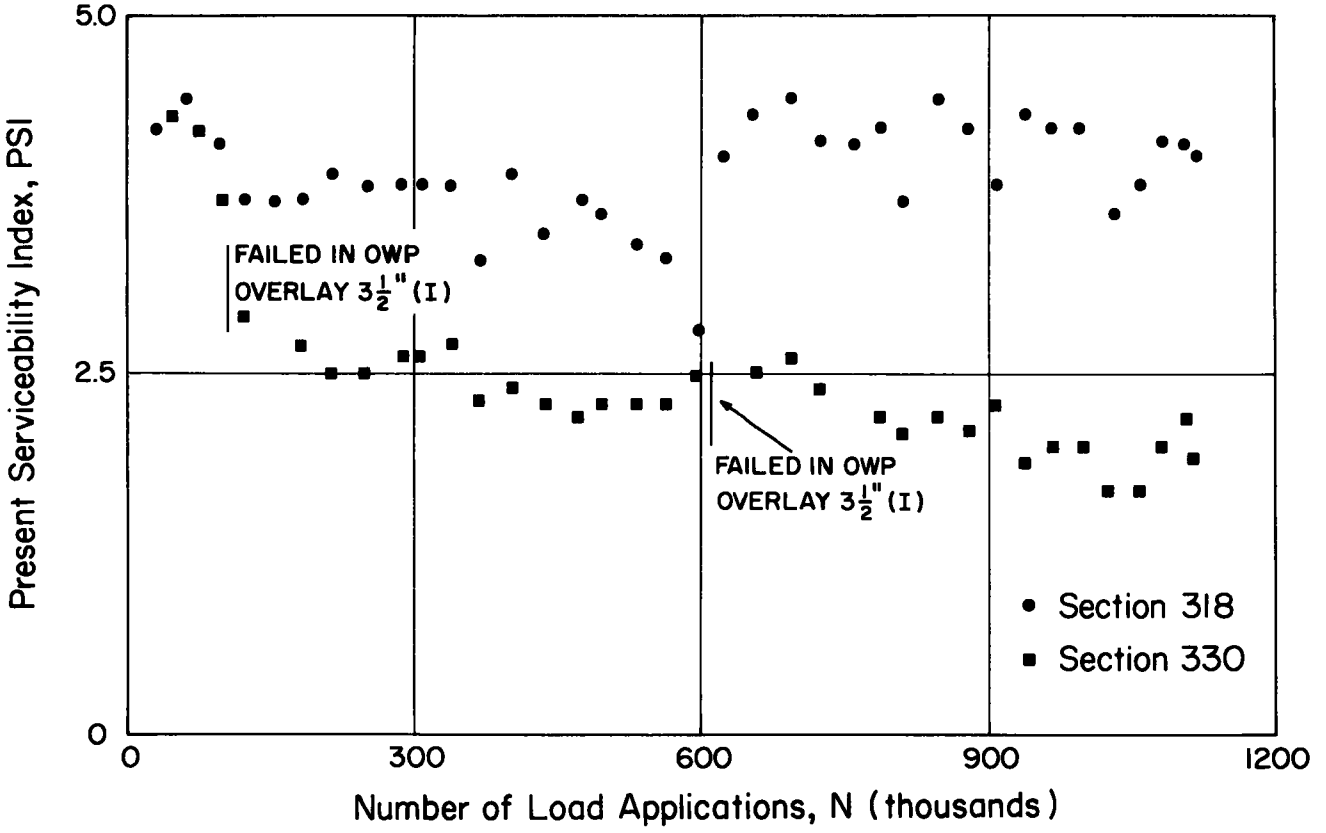

Figure A-6. Present serviceability index vs number of load applications; duplicability, sections 318 and 330.

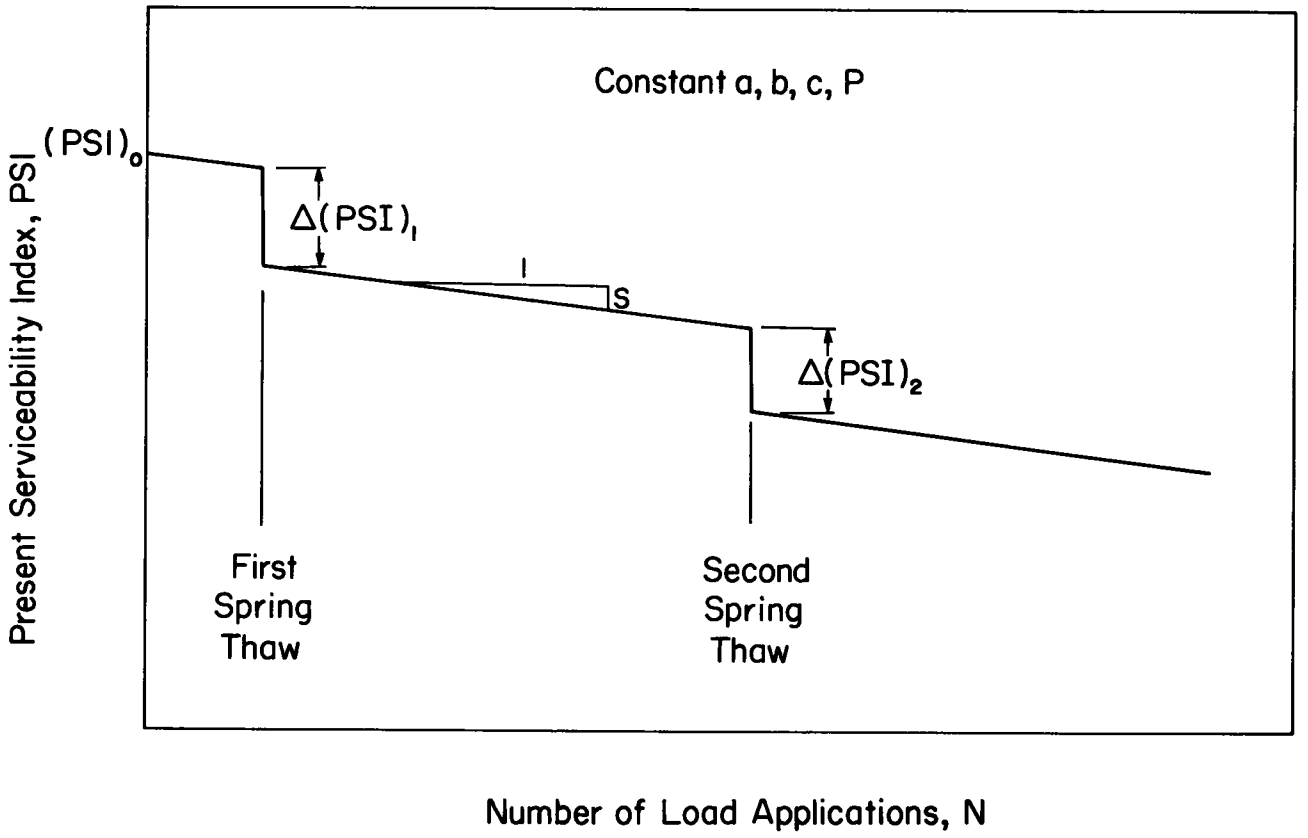

Figure A-7. Typical response trend for PSI vs N.

$$\mathrm{PSI}(N) = \mathrm{PSI}_0 - S\,N - \sum_{i=1}^{k} H(N_i)\Delta\mathrm{PSI}_i \quad \text{(A-2)}$$

in which PSI(N) is the PSI of a given pavement section after N applications of a given load, PSI_0, S, and $\Delta\mathrm{PSI}_i$ are as indicated in Figure A-7, $H(N_i)$ is the Heaviside unit step function, and k is the number of spring thaws which have occurred during N load applications. Because there is a correspondence between N and time, Eq. A-2 may readily be expressed in terms of time. It may be expected that the parameters $\Delta\mathrm{PSI}_i$ and S will be dependent on a, b, c, and P, while PSI_0 will be a function of construction specifications and quality of workmanship.

The initial PSI (that is, PSI_0) for all sections used in this analysis is plotted in Figure A-8. Such a plot indicates that, although the general range of the values for PSI_0 is between 3.3 and 4.6, most of the sections have an initial PSI value of either 4.1 or 4.2. Using a value of 4.2 for PSI_0 in Eq. A-2, there is an initial maximum deviation of 21 percent for sections having a PSI_0 of 3.3 before any effects of pavement deterioration are considered. Hence, the value for this term in Eq. A-2 may be left open, with the recommendation to substitute the actual value obtained. In many cases Eq. A-2 will be used to determine the expected change in PSI for a given period of time or a number of applications. For such situations the effect of PSI_0 will cancel out.

To investigate the dependence of S and $(\Delta\mathrm{PSI})_i$ on a, b, c, and P, the results of PSI versus N for the aforementioned

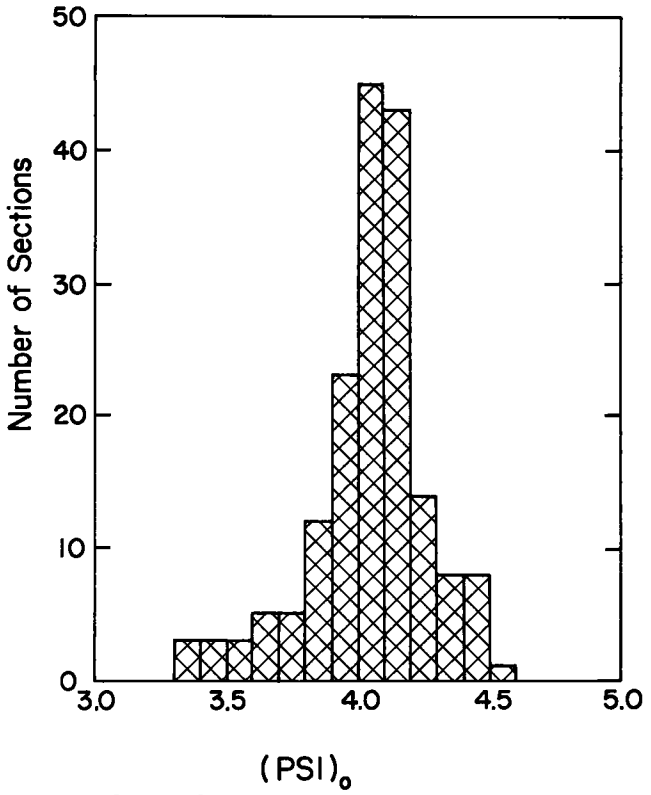

Figure A-8. Distribution of values of PSI_0.

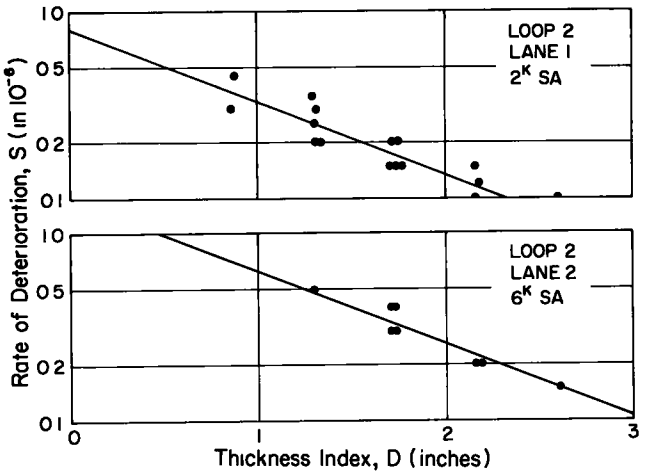

Figure A-9. Rate of deterioration vs thickness index for Loop 2 under various load conditions.

134 test sections were plotted and corresponding values for each parameter were obtained. If a systematic approach to the influence of each variable is pursued—for example, a—all other variables (that is, b, c, and P) must be maintained constant. If this is done, generally only about three points are available. Due to the magnitudes of variations previously discussed, three points were not found sufficient to objectively establish any trend and such attempts were abandoned. The thickness parameters (a, b, and c) were handled by combining them into a characteristic thickness index, D, according to

$$D = 0.44\,a + 0.14\,b + 0.11\,c \qquad (2)$$

as reported in HRB SR 61E (*2*). Figures A-9, A-10, and A-11 show the results of plotting deterioration rate, S, versus thickness index, D, for each lane of each traffic loop. In each of these lanes the applied axle load, P, is maintained constant and each point represents a test section. The data in each of these plots can be approximated by an equation of the form

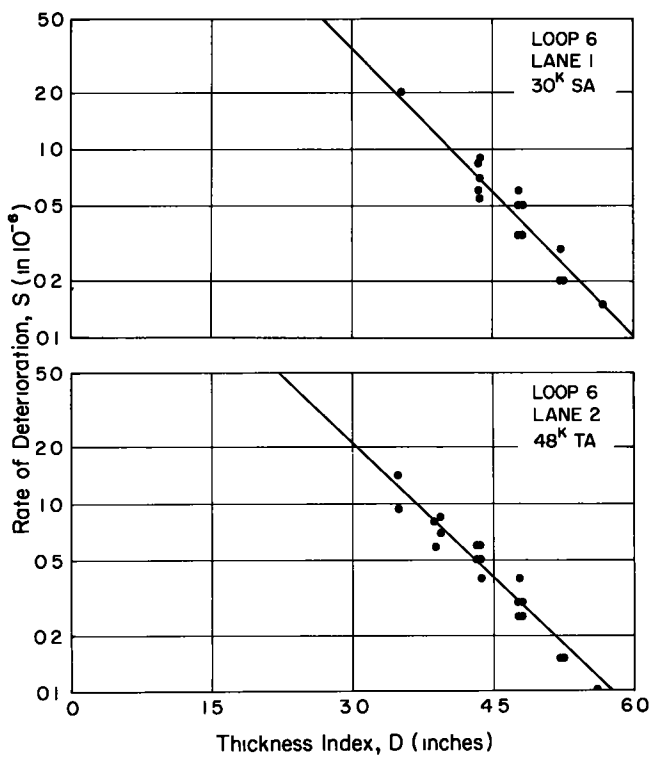

Figure A-11. Rate of deterioration vs thickness index for Loop 6 under various load conditions.

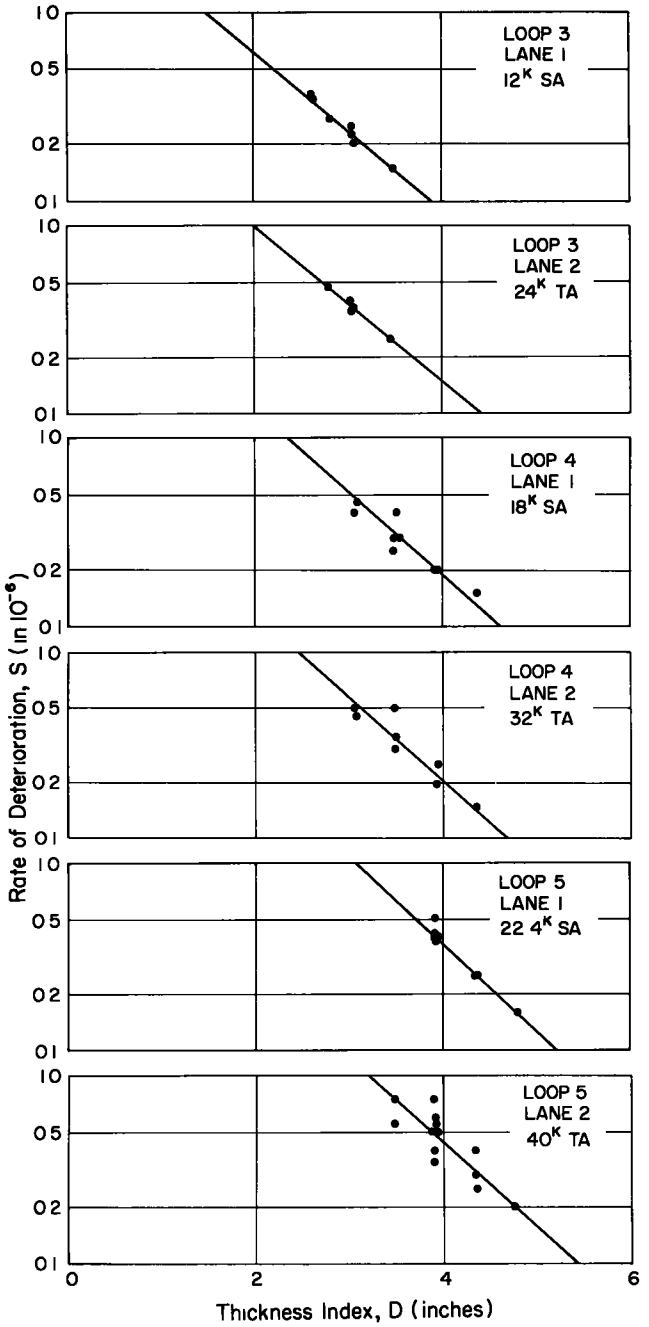

Figure A-10. Rate of deterioration vs thickness index for Loops 3, 4, and 5 under various load conditions.

$$S = \alpha\, 10^{-\beta D} \qquad (3)$$

where α and β are the intercept and slope, respectively, and dependent on P. Because both single-axle and tandem-axle loads are considered, some equivalency between the two must be established. This can be done by referring to Figure 22 of HRB SR 61E (*2*). Interpolation of the relationships given allows the tandem-axle loads utilized to be related to equivalent single-axle loads, as given in Table A-5. Using equivalent single-axle loads, P, the intercepts α and slopes β from Figures A-9, A-10, and A-11 (and expressed in Eq. 3) may be plotted as shown in Figures A-12 and A-13. The data in Figure A-12 may be approximated by

$$\alpha = 0.5 \times 10^{0.078P-6} \qquad (A\text{-}3)$$

and that in Figure A-13 may be written

$$\beta = 0.35 + 0.005P \qquad (A\text{-}4)$$

where the equivalent single-axle load, P, is expressed in kips. Substituting Eqs. A-3 and A-4 in Eq. 3 yields

$$S = 0.5 \times 10^{0.078P-6-(0.35+0.005P)D} \qquad (A\text{-}5)$$

A correlation plot for calculated and measured slopes, S, is given in Figure A-14.

The next phase of the study is directed at establishing a relationship for the first spring loss in serviceability, $(\Delta PSI)_1$, in terms of a, b, c, P, and N. The approach adopted is essentially similar to the approach utilized for S. In Figures A-15, A-16, and A-17 $(\Delta PSI)_1$ is plotted versus D. The general trend may be approximated by an equation of the form

$$(\Delta PSI)_1 = \gamma \times 10^{-\lambda D} \qquad (4)$$

where γ and λ are measures of the intercept and slope, respectively, and functions of P. Using single-axle loads and their equivalents (Table A-5), these intercepts and slopes may be plotted versus P, as shown in Figures A-18 and A-19, and the results can be approximated by

$$\gamma = 0.5 \times 10^{0.078P} \qquad (A\text{-}6)$$

TABLE A-5

EQUIVALENT SINGLE-AXLE LOADS

TANDEM-AXLE LOAD (KIPS)	EQUIVALENT SINGLE-AXLE LOAD (KIPS)
24	14
32	17.5
40	22
48	26

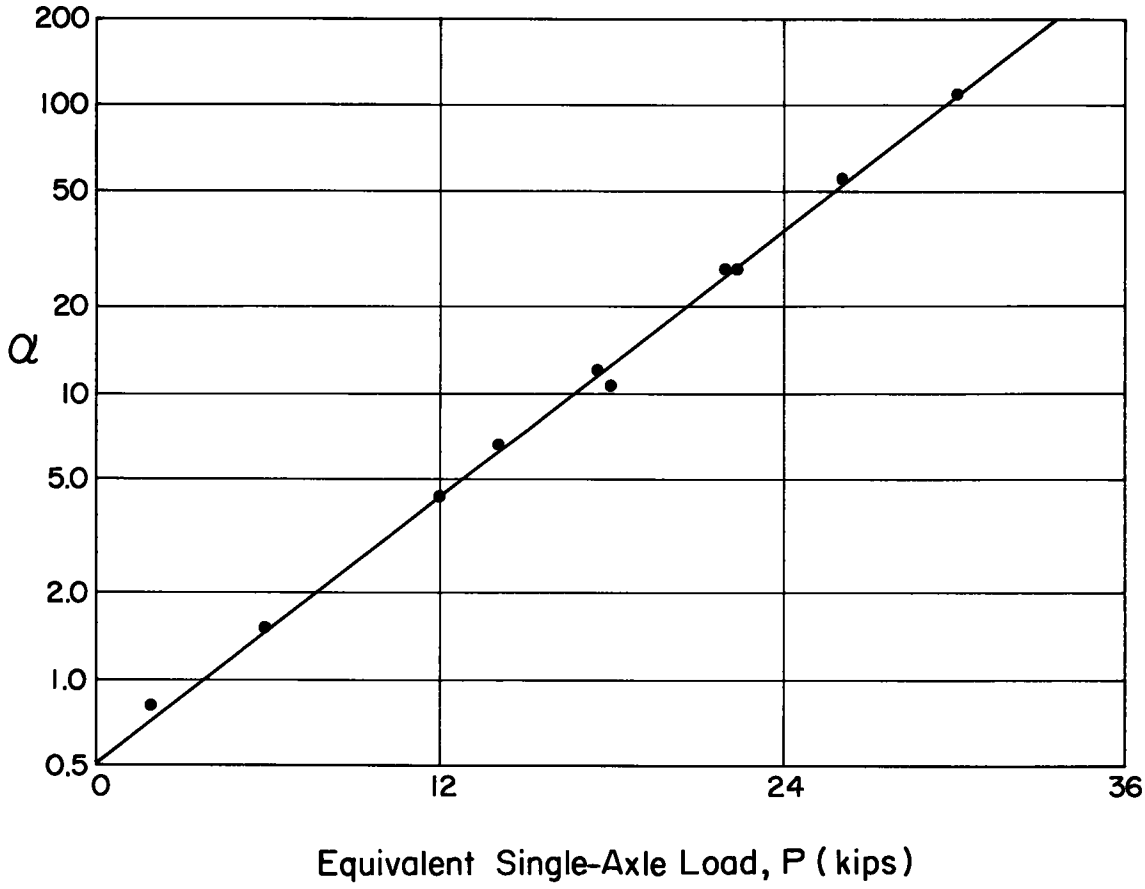

Figure A-12. Intercept parameter from rate of deterioration plot vs equivalent single-axle load.

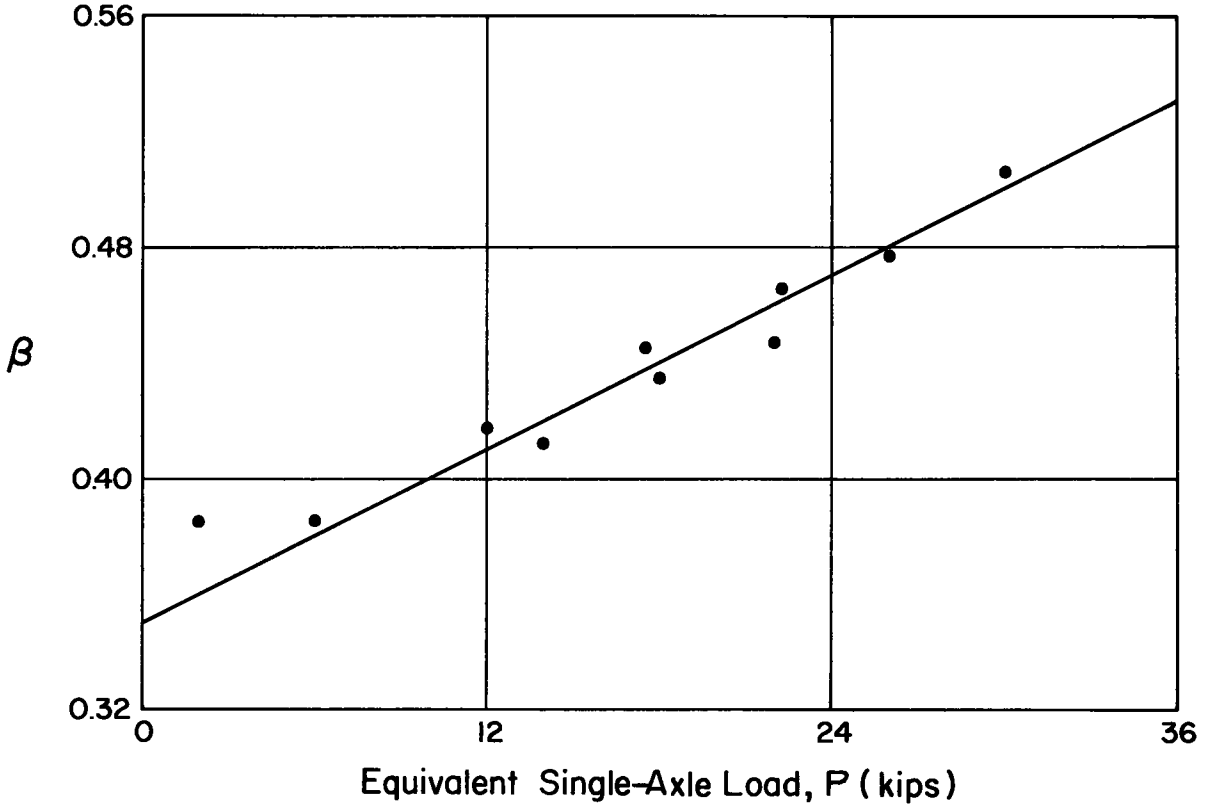

Figure A-13. Slope parameter from rate of deterioration plot vs equivalent single-axle load.

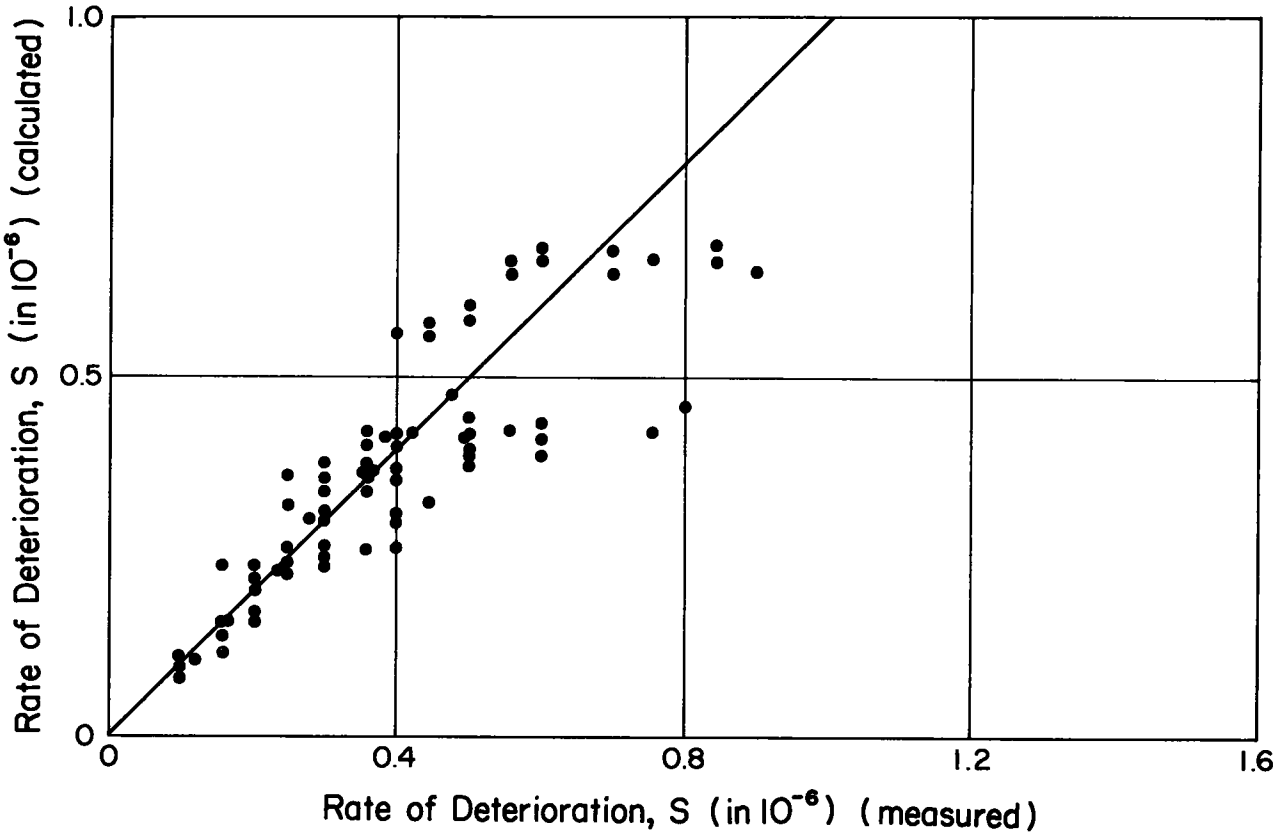

Figure A-14. Correlation plot of rates of deterioration, Eq. A-5.

and

$$\lambda = 0.24 + 0.0081\ P \qquad \text{(A-7)}$$

respectively, where P is expressed in kips. Substitution of Eqs. A-6 and A-7 in Eq. 4 gives

$$(\Delta PSI)_1 = 0.5 \times 10^{0.078P} \times 10^{-(0.24 + 0.0081P)D} \qquad \text{(A-8)}$$

The correlation between calculated and observed values for $(\Delta PSI)_1$ is shown in Figure A-20. Finally, substituting Eqs. A-5 and A-8 in Eq. A-2 gives, as a special equation for the performance of all the in-test flexible pavement sections (approximately 134) during the period until the second spring thaw,

$$PSI(N) = PSI_0 - 0.5(10)^{0.078P}\,[N(10)^{-[(0.35 + 0.005P)D + 6]} + H(140{,}000)(10)^{-(0.24 + 0.008P)D}] \qquad (10)$$

In Eq. 10, $H(140{,}000)$ is the Heaviside unit step function and has the value zero for $N < 140{,}000$ (that is, before the first spring thaw), and unity for $N \geqq 140{,}000$.

The effect of the Illinois environment for the first spring thaw has been taken into account in the parameter $(\Delta PSI)_1$, which is given by Eq. A-8. It may thereby be considered as a form of "climatic" or regional parameter which represents the annual adversity undergone by the pavement. To extend this concept to other localities, some estimate of this parameter must be made.

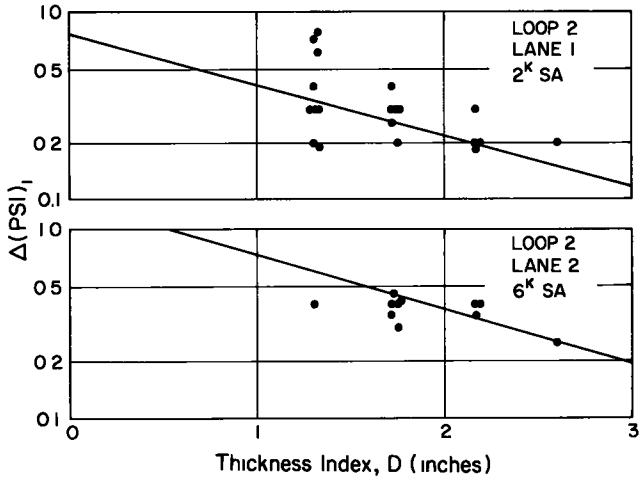

Figure A-15. First spring loss in serviceability vs thickness index for Loop 2 under various load conditions.

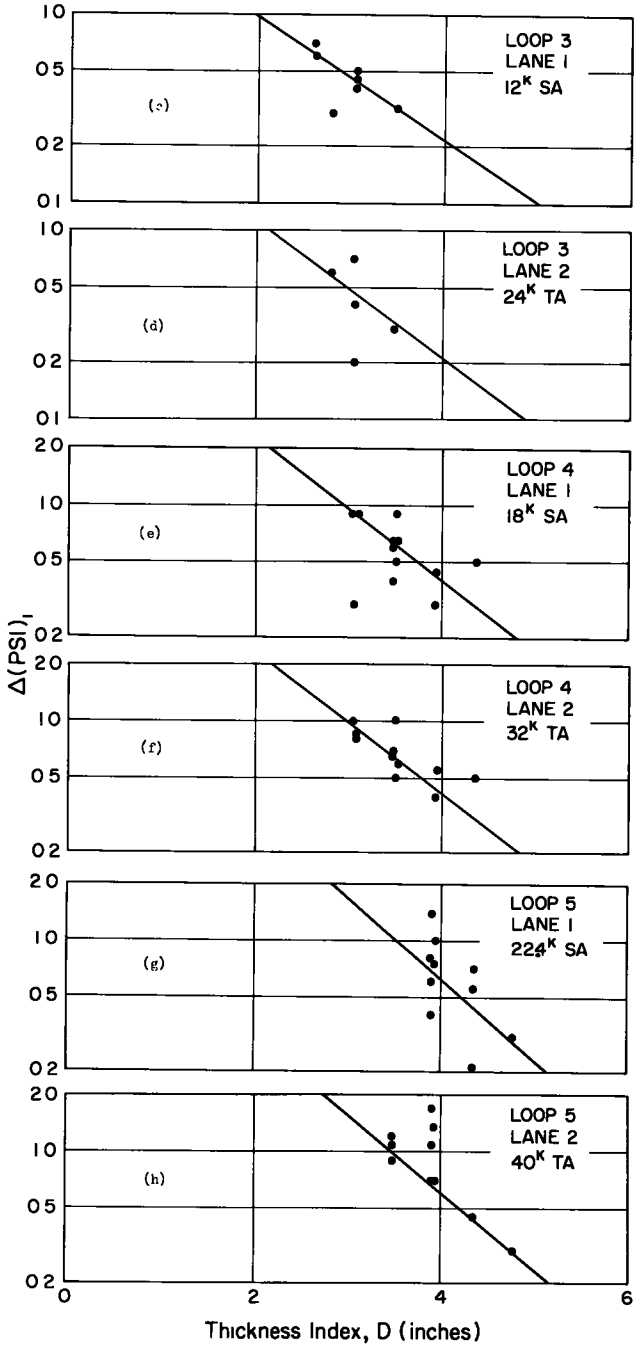

Figure A-16. First spring loss in serviceability vs thickness index for Loops 3, 4, and 5 under various load conditions.

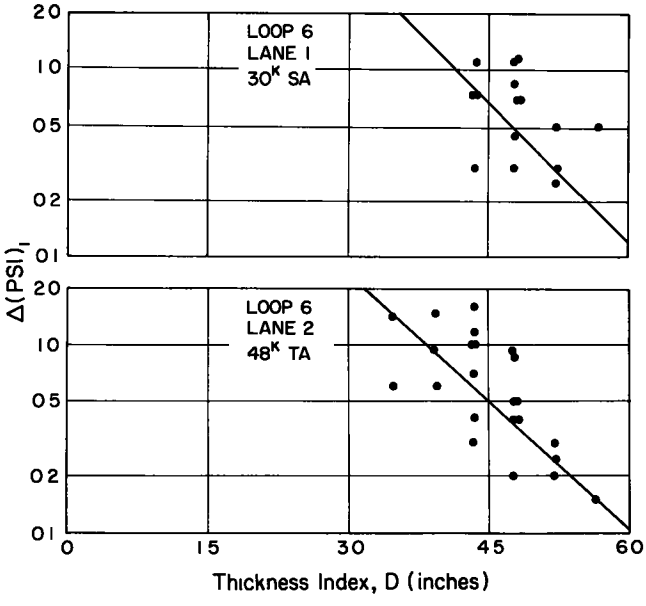

Figure A-17. First spring loss in serviceability vs thickness index for Loop 6 under various load conditions.

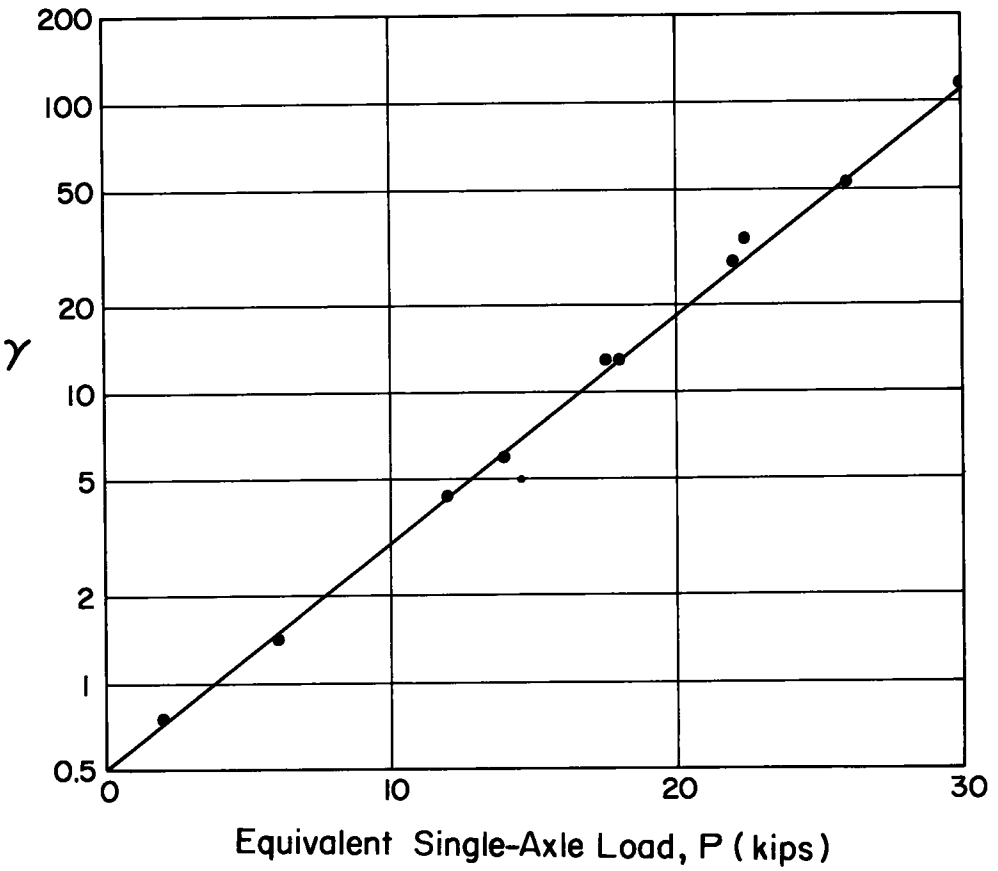

Figure A-18. Intercept parameter for first spring loss in serviceability vs equivalent single-axle load.

A relation similar to Eq. A-8 might be obtained for the second spring thaw. Unfortunately, the data accumulated at the AASHO Road Test indicate that a large portion (69 out of 134) of the test sections failed during the second spring. The remaining 65 sections were insufficient to formulate any reasonably conclusive trend, especially in view of the fact that many of these sections were overdesigned. In many cases, it was observed that the loss in serviceability during the second spring thaw is nearly the same as during the first spring thaw. For these sections, Eq. A-8 will yield a reasonable estimate of $(\Delta PSI)_2$. This is shown in Figure A-21, where the values of the loss in serviceability for the second spring thaw as given by Eq. A-8 have been correlated to the observed values of loss of serviceability for the second spring thaw for the 65 sections which endured throughout the duration of the Road Test.

Creep Speed Deflections

DEFLECTION MEASUREMENT

One of the objectives of deflection measurements is to determine the extent to which pavement deflections constitute an acceptable index of pavement serviceability. Although quasi-static rigid plate deflections of flexible pavements are considered in the following section, this section considers only Benkelman beam creep speed deflections.

The amount of deflection by a flexible pavement due to a given load is a measure of its structural capacity. Repeated deflections may cause pavement cracking and distortion as a result of fatigue or accumulated plastic deformation. Deflection measurements may be correlated with such quantitative measures as slope variance, rut depth, and roughness, and thus form the basis for evaluating the adequacy of existing pavements; also, several methods of flexible pavement design are based on limiting deflection criteria.

Insofar as structural adequacy is concerned, the primary purpose of measuring the deflection of a pavement is to obtain basic data relative to its stress-strain properties. Often, measurement of gross deflection at a pavement surface may not produce the desired results; additional factors, such as radius of bending and deflection of component layers, may override inferences drawn from total deflection. Ideally, pavement stress should be related to the ability of the paving materials to resist these stresses. However, due to the nonhomogeneous construction of a pavement section, stresses due to a load are both difficult to compute and difficult to measure. In contrast, measurement of a pavement deflection under load is relatively simple. Because stresses are related to strains, pavement deflections indicate to some extent the ability of the pavement to resist applied forces, or its load-carrying capacity. Deflection measurements are

only a tool which can aid in the formulation of concepts regarding pavement behavior. Like many other index-type quantities, they are subject to certain limitations and must be regarded simply as a means toward an end and not as an end in themselves.

A review of the literature reveals various attempts at correlating deflection measurement with pavement response. In addition to the thickness of the various pavement components and the applied load, it appears that the two major factors influencing the deflection of a flexible pavement are the condition of the subsurface components and the temperature of the surface. Williams and Lee (*21*) studied several high-type flexible pavements in Maryland and reported that outer wheelpath deflections were generally greater than corresponding inner wheelpath deflections. Also, attempts to correlate pavement deflections with subgrade soil surveys made prior to construction produced no definite trends and marked effects were caused by changes in climatic conditions. Finally, they were not able to correlate structural adequacy and total deflection. Hicks (*22*) concludes that a deflection survey is an excellent means of evaluating flexible pavement serviceability. Nichols (*25*) finds that deflections are more closely allied with compaction than pavement design characteristics. Walker et al. (*27*) observe that there is a direct relationship between total deflection and rutting, but no relation between total deflection and cracking. Dunlap and Stark (*24*) discuss techniques for measurement and correction of Benkelman beam deflections. Both the WASHO and the AASHO Road Tests have reported various correlations with pavement deflections, although several inconsistencies are noted.

One very effective analysis of performance-deflection relationships is given for the AASHO data in HRB SR 61E (*2*, pp. 108-112). In this analysis creep speed deflection is related to the number of weighted axle load applications under a given load required to reduce the PSI to some predetermined value. A correlation is made using both the readings from the fall of 1958 and the spring of 1959. This indicates that deflection studies are useful when approached with a very limited scope, but become quite difficult when attempting too broad a correlation. For example, in the correlation shown in HRB SR 61E the statistical coefficients and resulting curves obtained in the correlations involving the different seasonal deflections are different, as expected.

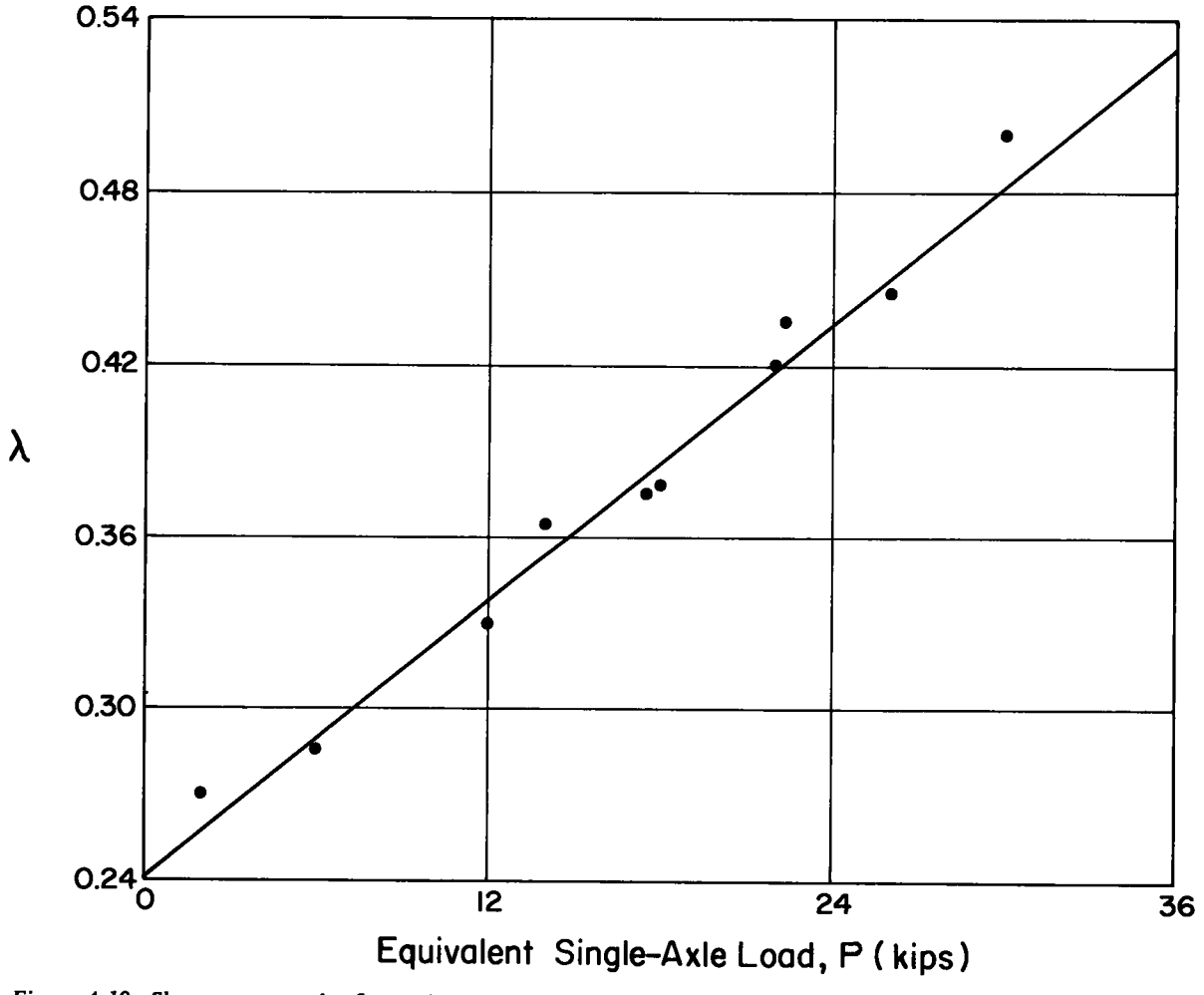

Figure A-19. Slope parameter for first spring loss in serviceability vs equivalent single-axle load.

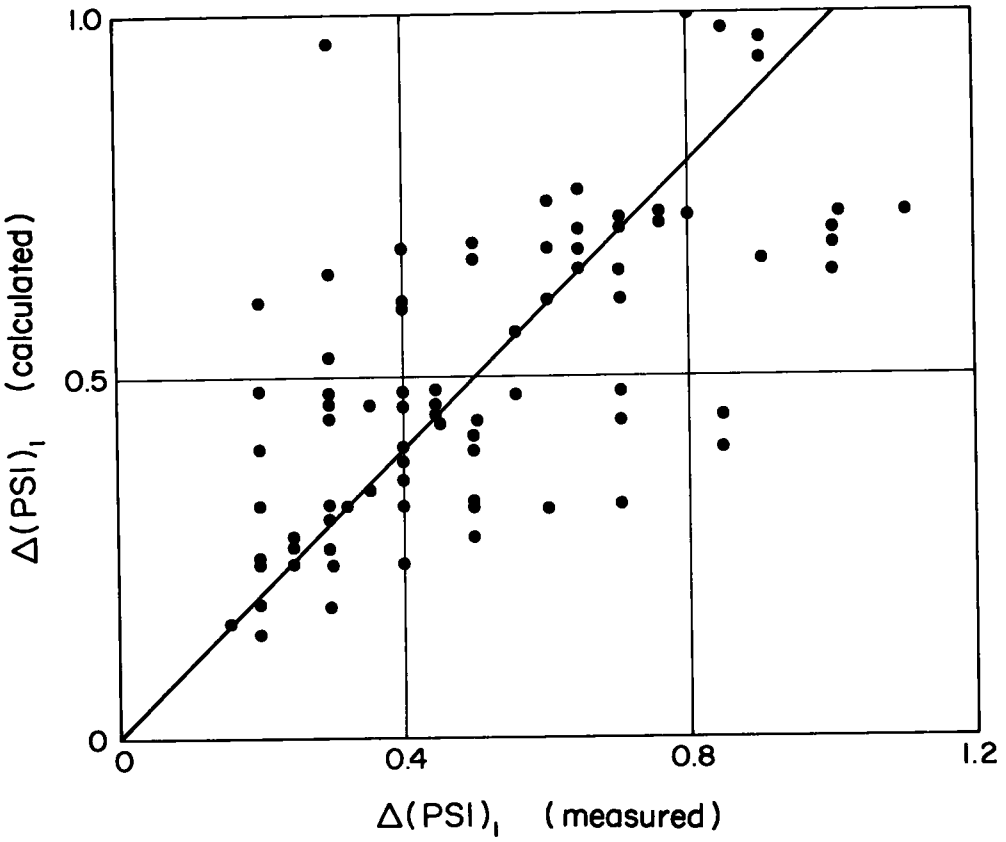

Figure A-20. Correlation plot of first spring loss in serviceability, Eq. A-8.

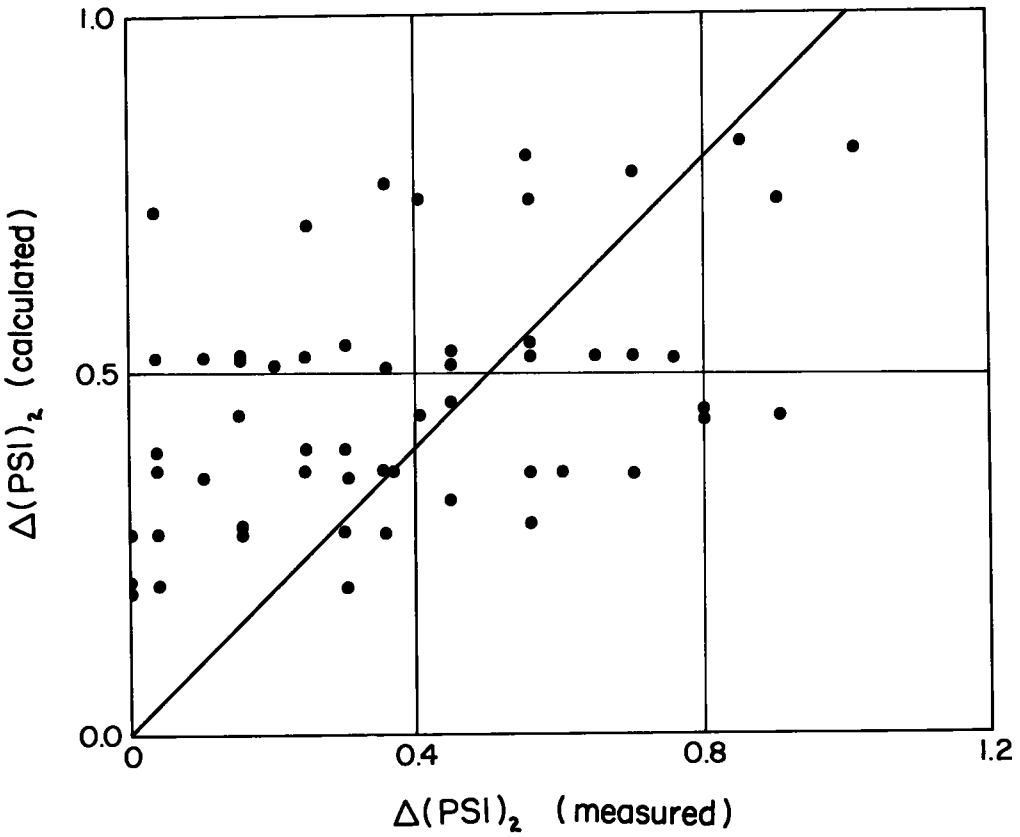

Figure A-21. Correlation plot of second spring loss in serviceability, Eq. A-8.

Furthermore, it might be anticipated that use of deflections at some other time would produce still different coefficients and curves. The surfacing temperature dependence of pavement deflection is also studied in HRB SR 61E (*2*, pp. 101-108). Also included are deflection-load (pp. 96-100) and deflection-speed (pp. 100-101) relationships.

Because normal deflections could not be obtained with the Benkelman beam under tandem-axle loads, use of single-tandem relationships, as given in Table 5, must be employed.

Data obtained from the trench studies on Loop 1 (the nontraffic loop) were used to develop correlations between Benkelman beam deflection and the results of the plate load tests. A sample of these correlations is shown in Figure 99 of HRB SR 61E (*2*); Sebastyan (*30*) has found essentially the same type of correlation. Relations were found to be essentially linear. The following section concerns itself with deflections under rigid bearing plates.

DEPENDENCE ON INDEX DAY

The problem of correlating deflection data becomes even more difficult than the correlations already attempted. The reason for this is as follows. Performance indices such as PSR (or PSI), *SV*, *RD*, and *R* depend only on a subjective observation or objective measurements made on the surface of any given pavement at any given time. Although it is quite true that the structural composition of the pavement, *a*, *b*, and *c*, vehicle wheel load, *P*, and number of applications, *N*, definitely play their role in influencing the condition of the pavement at any given time, such factors are not directly considered in the determination of the index. This is not the case with deflection measurements. The amount of deflection attained at any given time is dependent not only on the condition of the pavement as affected by past history but also by immediate considerations such as the structure of the section, strengths of various components, and applied load under which the deflection is measured. Thus, although for an index like PSI a relatively linear relationship is obtained throughout the year except for spring thaws, such is not to be expected for deflection measurements. Rather it is to be expected that such measurements will fluctuate as climatic conditions change due to the seasons of the year. This is precisely what happens in virtually all cases. A typical plot of Benkelman beam deflection, δ, versus calendar day, CD, is given in Figure A-22. Calendar day, CD, is a four-digit time identification code that increases by one every 24 hours (July 1, 1956 = 0000; November 3, 1958 = 0855; November 30, 1960 = 1613). Controlled traffic was begun on calendar day 0855 and stopped on calendar day 1613. The relationship between CD and *N* is shown in Figure A-23. One type of curve fit which becomes immediately obvious in Figure

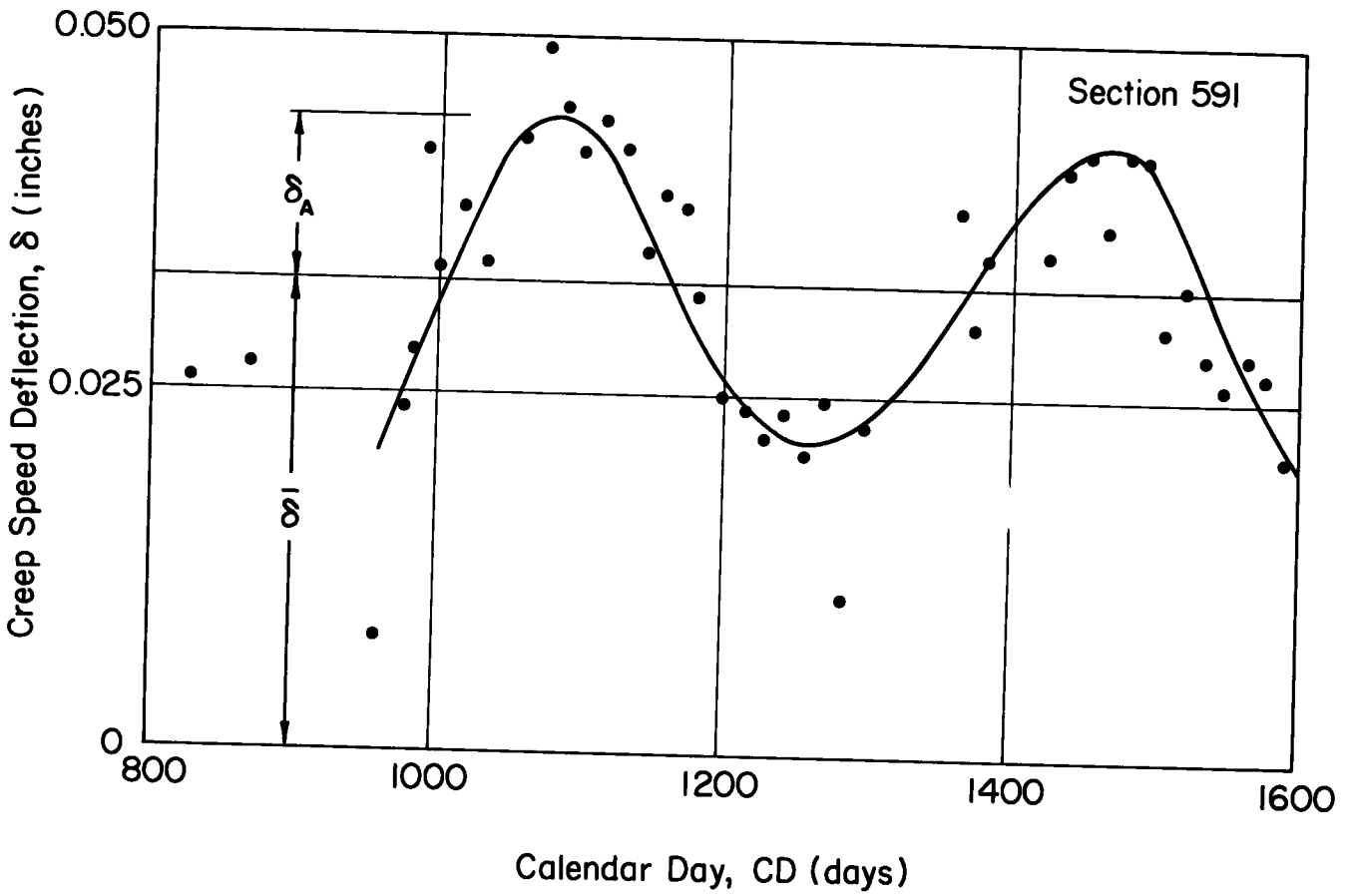

Figure A-22. Typical plot of creep speed deflection vs calendar day.

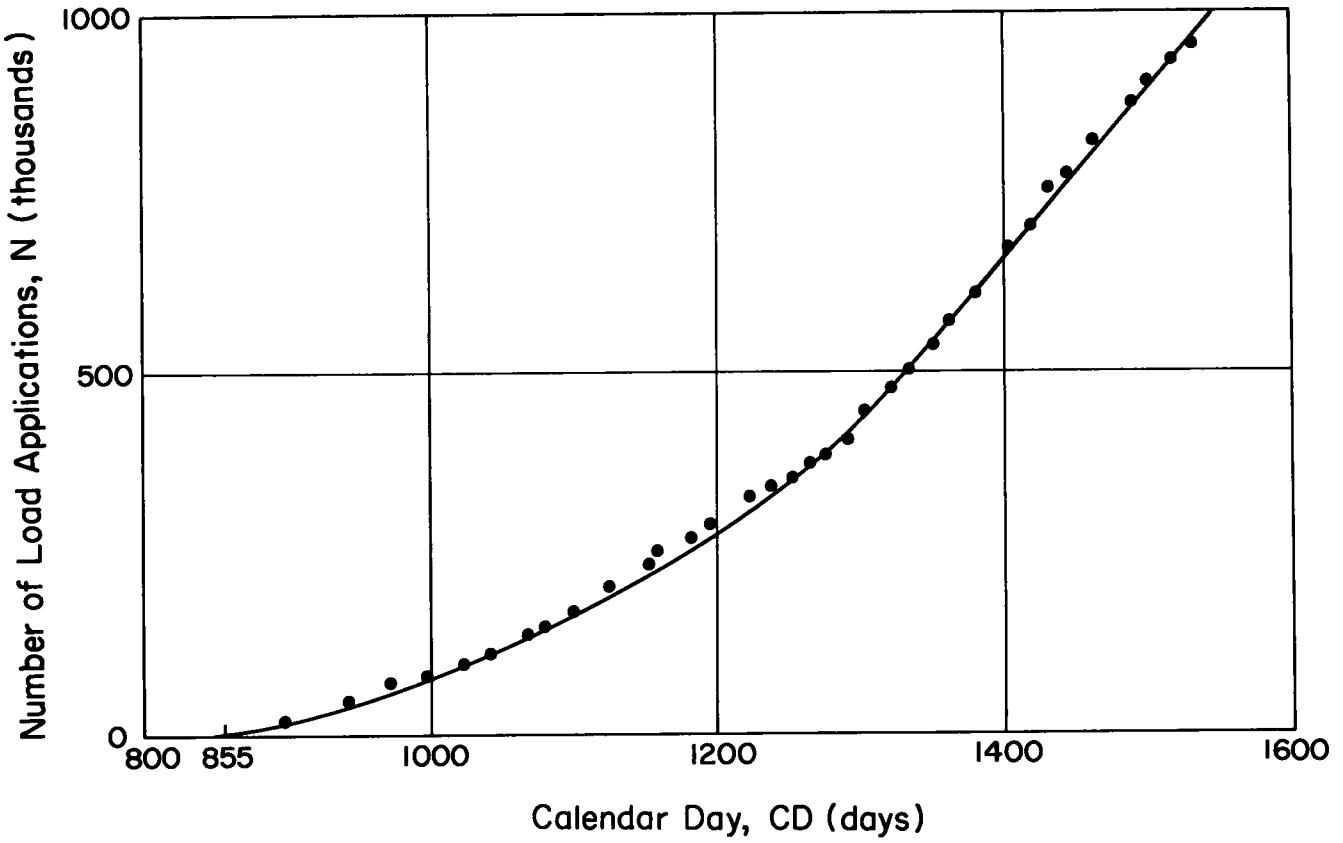

Figure A-23. Number of load applications vs calendar day.

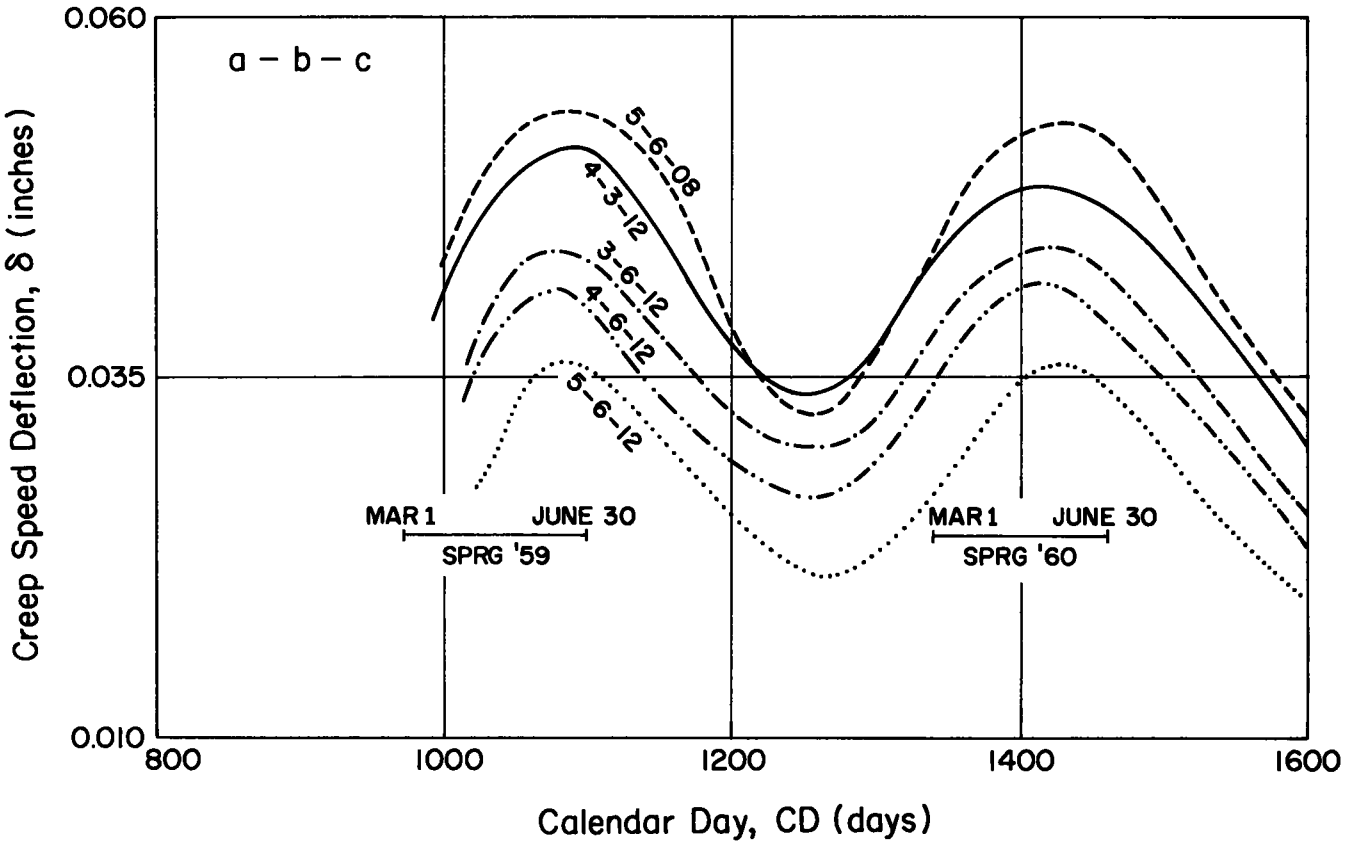

Figure A-24. Creep speed deflection vs calendar day for various pavement thicknesses and axle loads.

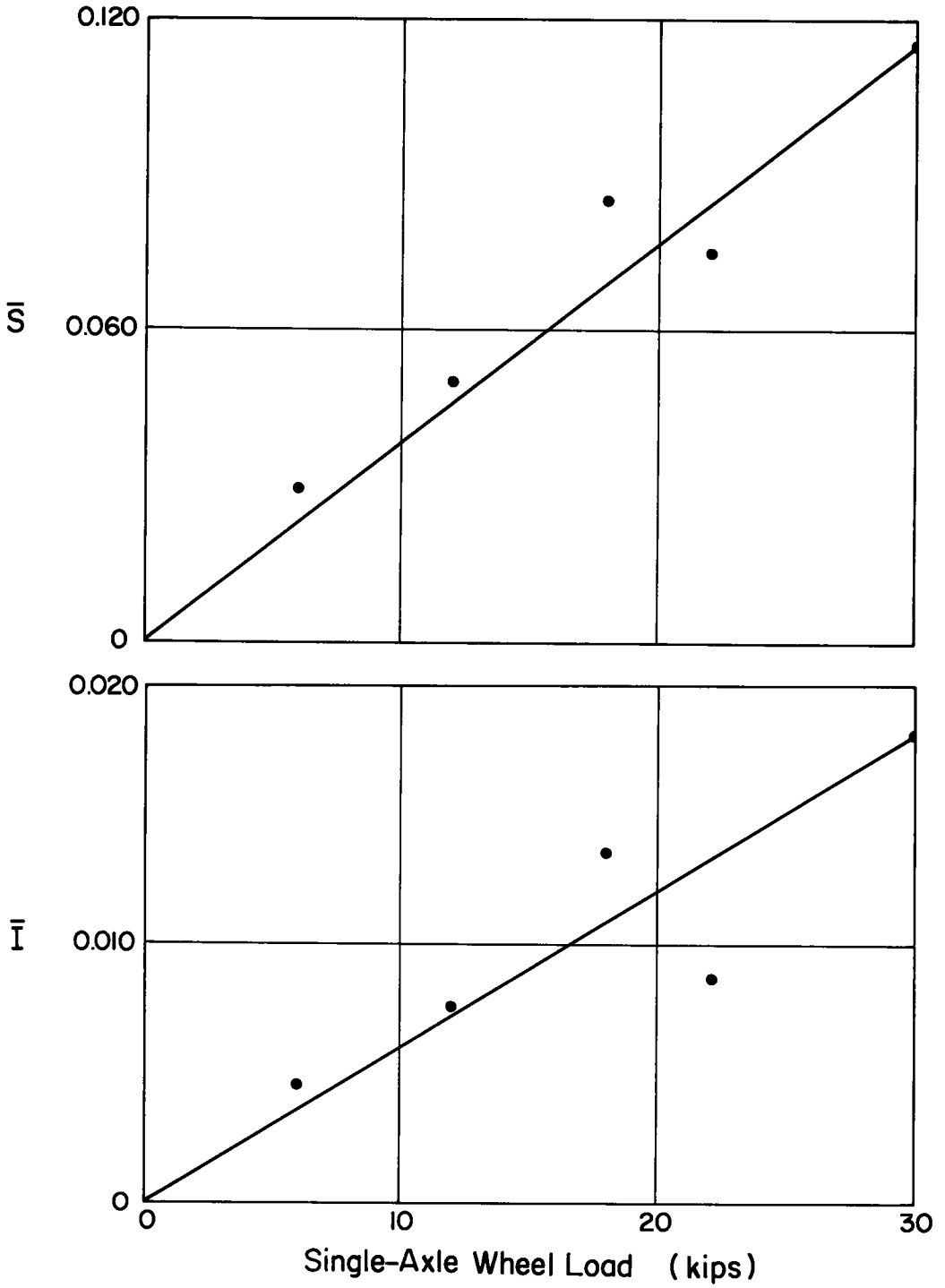

Figure A-25. Mean creep speed deflection parameters vs single-axle wheel load.

A-22 is that of a sine wave with a period of one year oscillating about a displaced base line; such an equation can be written as

$$\delta = \bar{\delta} + \delta_A \sin\left(\frac{2\pi}{365}[t - t_0]\right) \qquad (19)$$

where $\bar{\delta}$ indicates the level of the base line, δ_A is the amplitude of the deflection wave, t is the time in calendar days measured for the AASHO project, and t_0 is any time in calendar days when the increasing deflection wave crosses the base line. For the two-year test period of the AASHO Road Test, most of the base lines were very nearly horizontal with some having a slight tendency toward a gentle negative slope. Although it might be expected that the pavement has a tendency to weaken as time (or number of load applications) increases, such trends can not be readily deduced from the Road Test data. In addition to the slight negative slope noted in several cases, the amplitude of the

wave seems to remain constant or to decrease very slightly for the two cycles observed. All of the observed deflection data for each lane were made under the same axle load as that of the vehicle operating in that lane. Normal deflections can not be obtained with the Benkelman beam under tandem-axle loads. It would be expected that the intercept and slope of the base line, as well as the amplitude of the sine wave, would be dependent on *a, b, c,* and *P*. If the amplitude did indeed show a tendency to decrease with time (or number of applications), this could probably be taken into account with a "damping" or attenuation factor, such as e^{-kN}. There is insufficient evidence in the AASHO data to recommend that such an approach be followed, but subsequent data may exhibit such tendencies.

The relative behavioral patterns of several different pavement sections may be seen in Figure A-24, where average curves have already been sketched through the data. In general, at least two curves have all variables (*a, b, c,* and *P*) except one in common; hence, the relative effect of that one variable can be observed. It must be remembered that the deformations were determined under an axle load equal to the axle load utilized in the traffic study. Deflections would vary, of course, with the load utilized.

A review of the data represented in Figures A-22 and A-24 suggests that the measurement of deflection alone is not a sufficient criterion to deduce any information relating to pavement serviceability and performance. The wide variation with seasons necessitates a consideration of when the measurement was made. This is intended to imply that deflection measurements may be useful in evaluating pavement performance, but only when additional information is properly considered.

DEPENDENCE ON LOCAL FACTORS

Prior to any attempt to determine a relationship between δ and the various local factors considered (*a, b, c,* and *P*), the assumptions are made that the base line is horizontal and the amplitude is constant. Evidence available for the two-cycle duration of this study is insufficient to suggest any other realistic conclusions regarding these parameters; however, the foregoing assumptions are subject to modifications as additional data become available. Excluding isolated exceptionally high or low values, maximum and minimum deflections in lane 1 of Loops 2 through 6 were recorded for the two-year duration of the project. The double amplitude is designated $2\delta_A$ and taken as the maximum minus the minimum deflection; the mean deflection is symbolized by $\bar{\delta}$ and taken as the average of the maximum and minimum deflections. The level of the base line is established by the mean deflection.

Values for $\bar{\delta}$ and $2\delta_A$ are plotted versus the thickness index, *D*, for each load and approximated by straight lines whose equations are of the general form

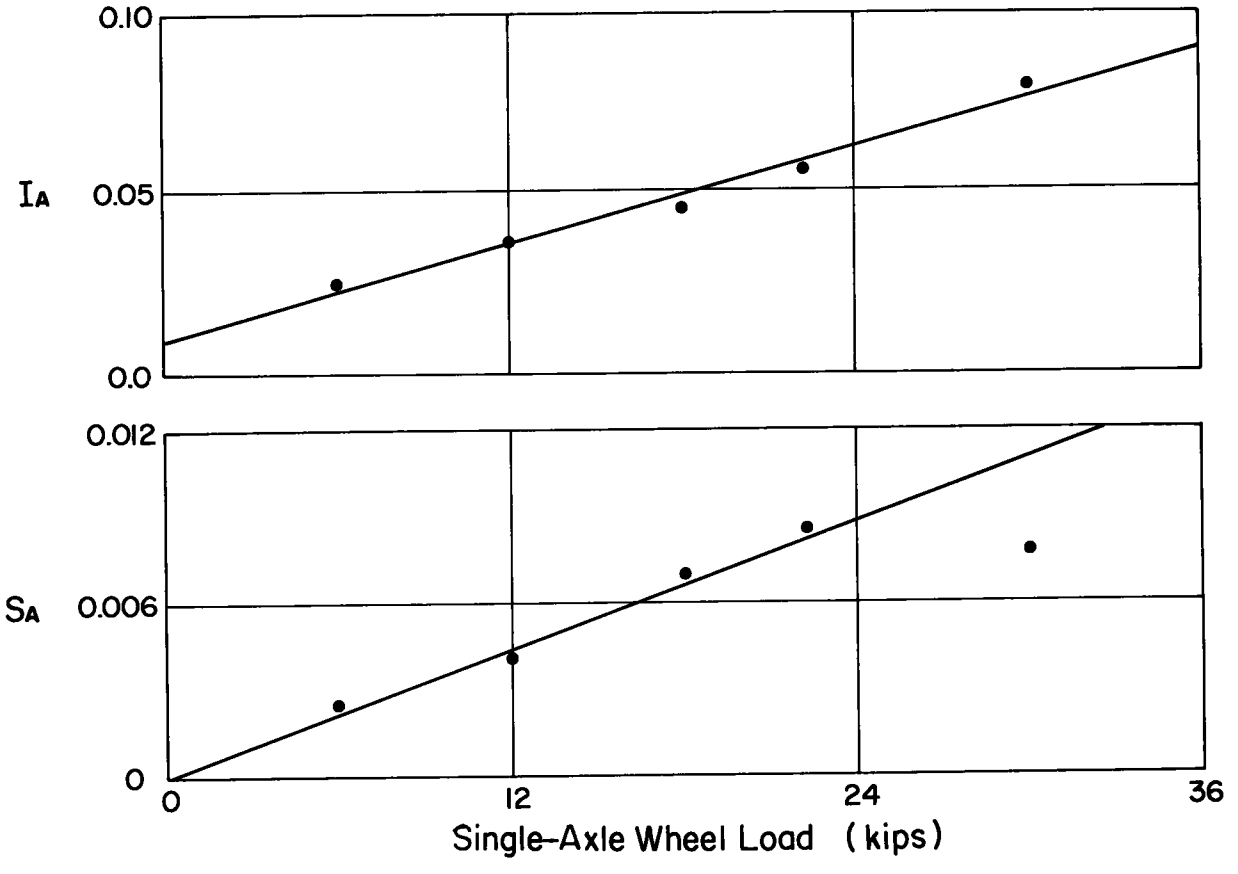

Figure A-26. Amplitude creep speed deflection parameters vs single-axle wheel load.

$$\bar{\delta} = \bar{I} - \bar{S}D \qquad (20)$$

and

$$2\delta_A = I_A - S_A D \qquad (21)$$

The intercepts and slopes, respectively, of Eqs. 20 and 21 are functions of applied load, P. These relationships are shown in Figures A-25 and A-26, where each point pertains to the intercept or slope for a given applied load. The following approximations may be written as

$$\bar{I} = 0.0038P \qquad \text{(A-9)}$$

$$\bar{S} = 0.0006P \qquad \text{(A-10)}$$

$$I_A = 0.01 + 0.0023P \qquad \text{(A-11)}$$

and

$$S_A = 0.00037P \qquad \text{(A-12)}$$

where P is expressed in kips. Substituting Eqs. A-9 and A-10 in Eq. 20, and Eqs. A-11 and A-12 in Eq. 21, gives

$$\bar{\delta} = [0.0038 - 0.0006D]P \qquad \text{(A-13)}$$

and

$$2\delta_A = 0.01 + [0.0023 - 0.00037D]P \qquad \text{(A-14)}$$

Finally, substitution of Eqs. A-13 and A-14 in Eq. 19 yields

$$\delta = P[0.00038 - 0.0006D] + \left\{ P\left[\frac{0.005}{P} + 0.0011 - 0.00018D\right] \sin\left[\frac{2\pi}{365}(t - t_0)\right]\right\} \qquad (27)$$

which is the developed expression for the relationship between creep speed deflection, pavement structure, and applied load for any given time during the test traffic period of the AASHO Road Test.

Overlay Sections

Nearly 50 percent of the 288 flexible pavement sections failed early during the first spring season (spring 1959) and each was rebuilt to carry the test traffic. However, the performance of only about one-third of these sections was observed. In another group of approximately 60 sections structural deterioration was slower and these sections were overlaid as their PSI dropped to around 1.5 in either wheelpath during the second spring thaw (spring 1960). Observations of their performance were continued as for the in-test sections.

An overlay consisted essentially of applying a 2- to 3½-in. thick covering of asphaltic concrete over the surface of the existing pavement. Additional information relating to the details of the overlays may be found in HRB SR 61E

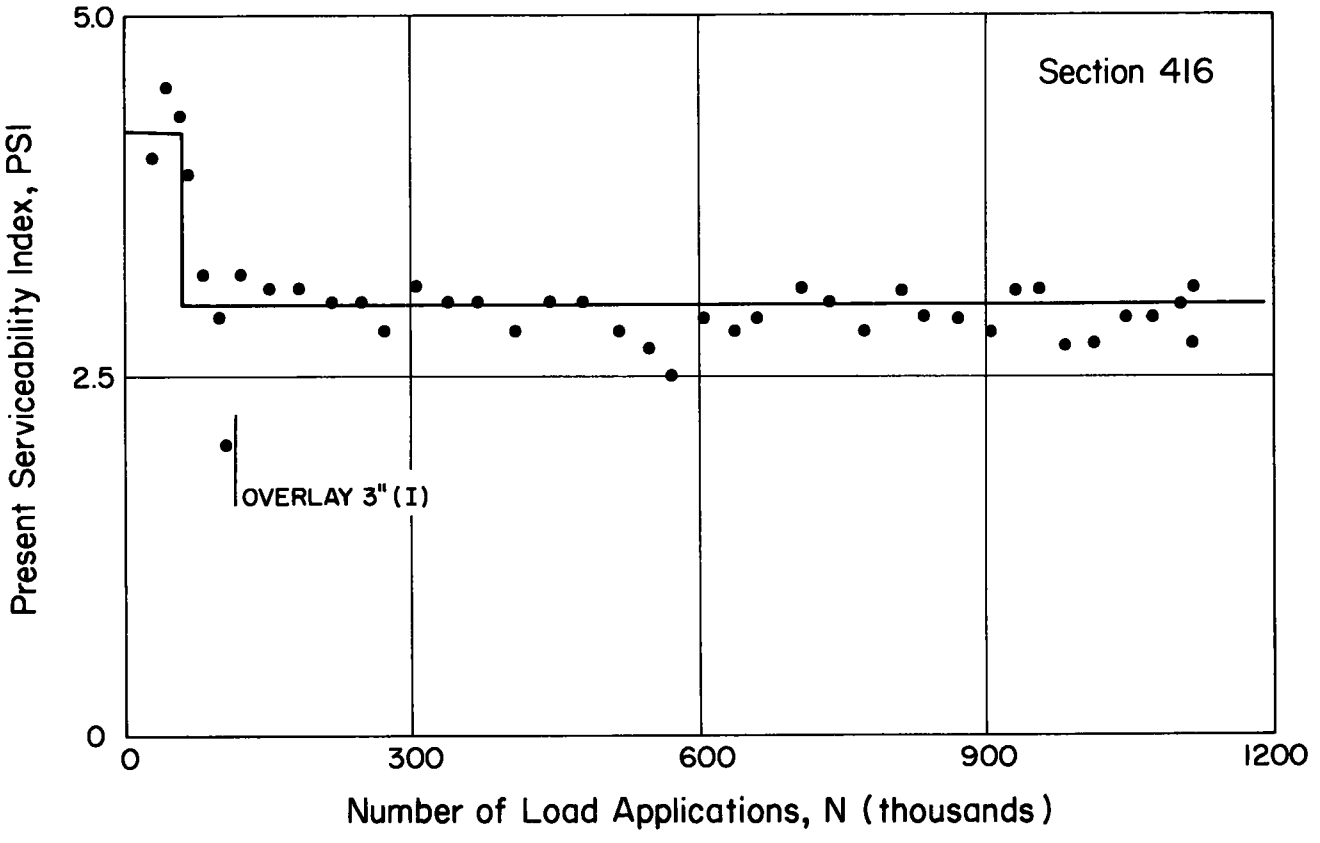

Figure A-27. Present serviceability index vs number of load applications, overlay section.

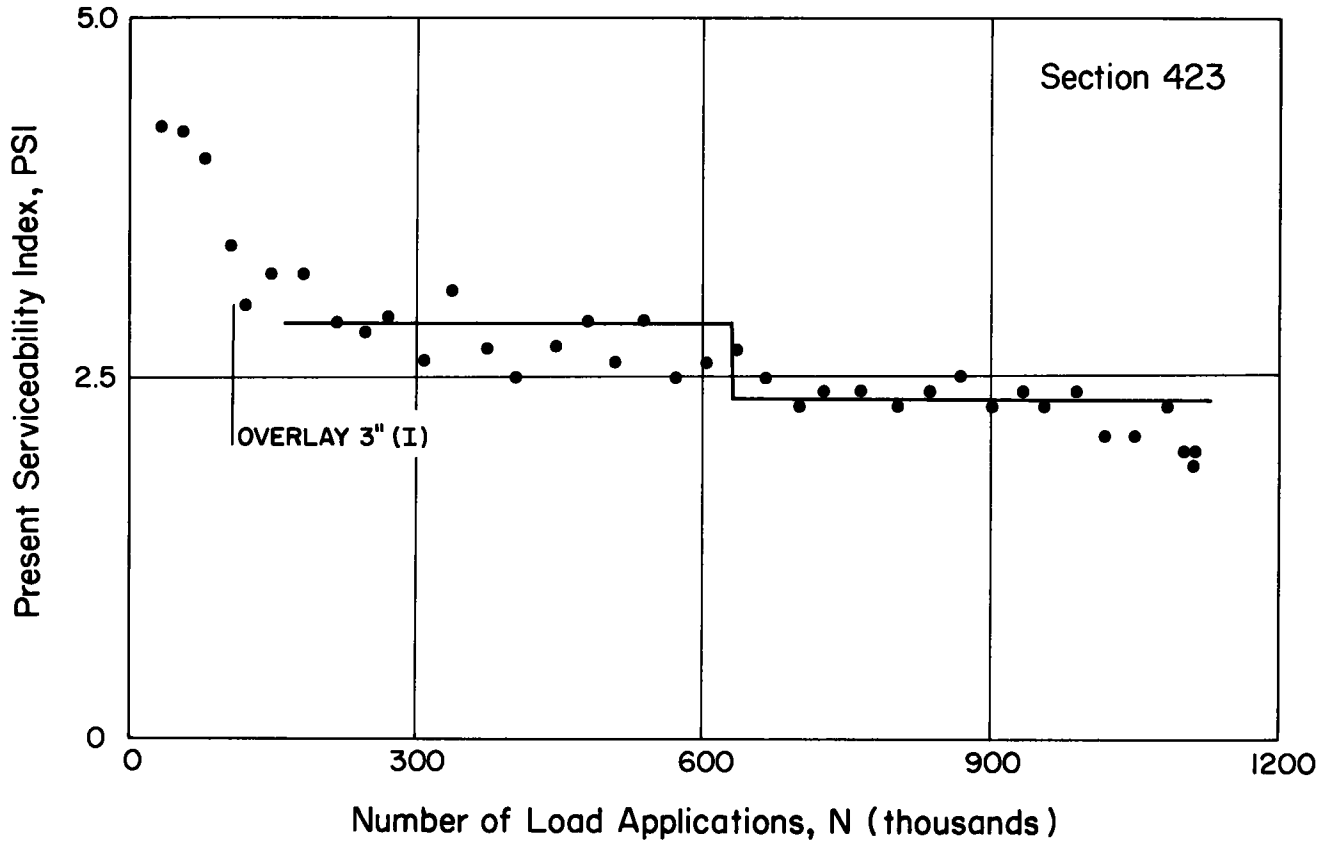

Figure A-28. Present serviceability index vs number of load applications, overlay section.

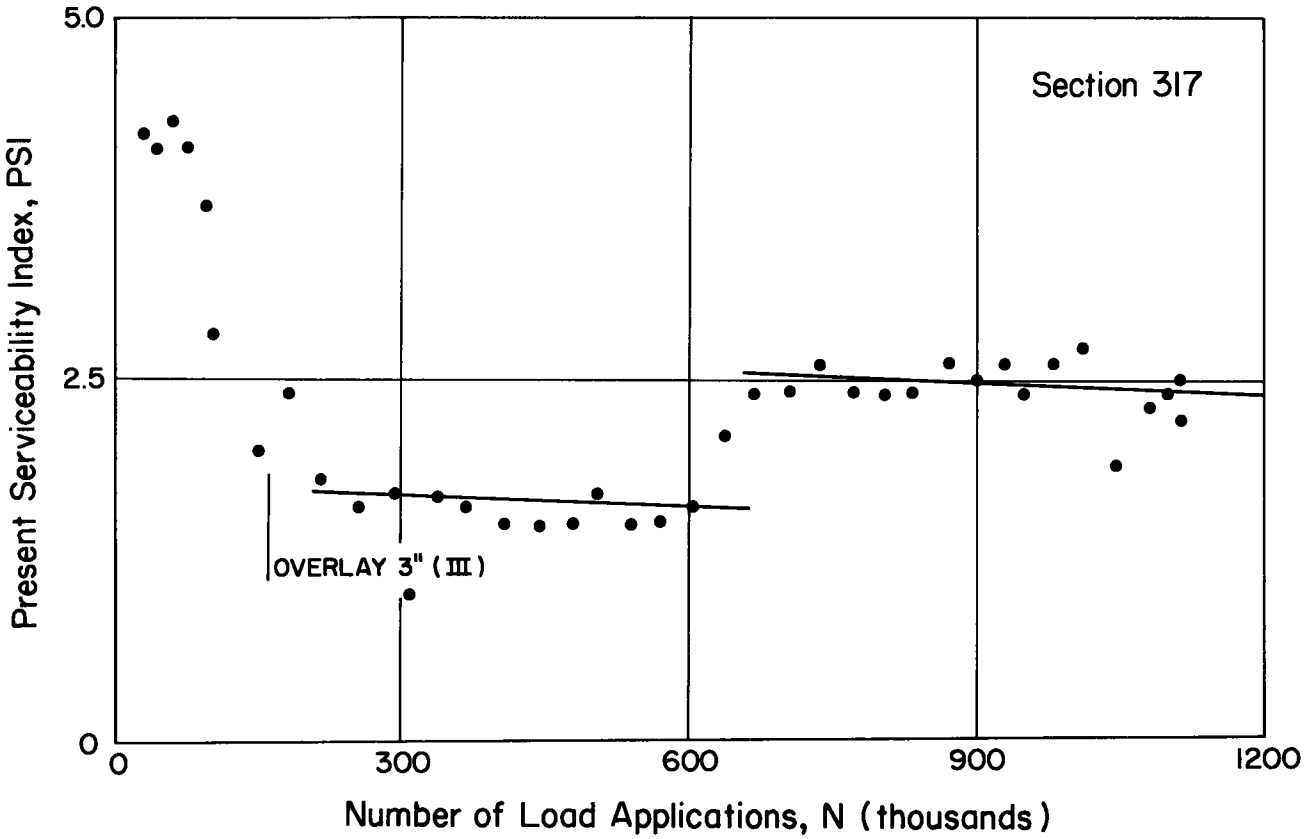

Figure A-29. Present serviceability index vs number of load applications, overlay section.

(*2*). Attempts to analyze the performance of such sections were not successful. In the present study most of the sections (overlaid after either the first or the second spring) show almost no effect of number of load applications. For the case of most sections which failed and were overlaid during the first spring, the PSI remained nearly constant during the second spring and throughout the test program, as shown in Figure A-27. However, some of the overlaid sections do show a drop in PSI during the second spring thaw, as indicated by Figure A-28, while others, as shown in Figure A-29, demonstrate an unexplained increase in PSI. Because the performance evidence of overlaid sections was so inconsistent, it was difficult to find a descriptive expression.

However, approximately ten overlay sections in lane 1 of Loop 6 showed a slight deterioration in performance with number of applications. Plotting slopes for each design versus the thickness index indicates that a performance relationship similar to that of the in-test sections may exist. The overlay thickness of 3 in. was constant for all the sections of this lane and seemed to make no difference in the performance response despite the fact that this additional overlay constituted a much larger addition to some sections than to others. Data were available for only three sections of lane 2 of Loop 6 and data from the other loops indicated no consistent pattern. Hence, any conclusion would be premature and purely speculative.

APPENDIX B

DEVELOPMENT OF BEARING PLATE TEST-LOCAL FACTORS RELATIONS

TEST FACILITIES AT HYBLA VALLEY

The development of relations between bearing plate test results and local factors was attempted using the results of the Hybla Valley Field Test program, run on an experimental test track near Hybla Valley, about 4 miles south of Alexandria, Va. Test sections consisted of various combinations of asphaltic concrete and base course thicknesses ranging from 0 to 12 in. and 0 to 24 in., respectively. These pavement sections were constructed with great care on a minimum embankment of 5 ft of uniform A-7-6 soil (AASHO Classification—1949), and every precaution was exercised to insure uniformity of thickness, compaction and composition of the various component layers. The soil used in the embankment was secured from a previously prospected area, and a high degree of uniformity, both in composition and condition, was obtained. The first stages were completed in 1946, but some sections were not placed until 1949.

The pavement sections may be classified into two categories according to the function they served. Auxiliary sections were constructed in advance of the test track to provide a test area for the large amount of preliminary work that was contemplated in developing test equipment, techniques and methods. These tests were primarily exploratory in nature rather than for the purpose of obtaining quantitative data. The main sections were built on the north and south tangents of the oval test track and provide the test area for obtaining the quantitative data analyzed herein.

DESCRIPTION OF TEST PROCEDURES

There are innumerable possible procedures for conducting static load tests. For any given pavement section the various controllable factors that may affect the results of tests of this type include the magnitude of the load and the manner in which it is applied, the number of applications and releases of a given load, the duration of each load application and release, and the size of the bearing plate. A further discussion of such variables has been given by Kondner and Krizek (*31*). The data presented by Benkelman and Williams (*4*) were obtained by the use of four different load-test procedures; namely, the incremental, the incremental-repetitional, the accelerated, and the repetitional. The vast majority of the tests was made with the accelerated and repetitional procedures. Further details regarding these test procedures can be obtained from HRB SR 46 (*4*).

DIMENSIONAL ANALYSIS

Because of the complex properties of the various pavement materials and the complicated interaction of these various layers with the loads being supported, the use of nondimensional techniques in both model and prototype research investigations of pavement problems seems to offer definite advantages. This study not only provides an analysis of a portion of the Hybla Valley test data, but it also illustrates an application of nondimensional techniques in the field of pavement design. One previous example has been reported by Kondner and Krizek (*6*).

The methods of dimensional analysis as used to determine relationships between physical quantities may be briefly summarized as follows. If there are m physical quantities containing n fundamental units which can be related by an equation, then there are $(m-n)$, and only $(m-n)$, independent nondimensional parameters, called π terms, which are arguments of an indeterminate homogeneous function F; that is,

$$F(\pi_1, \pi_2, \pi_3, \ldots, \pi_{m-n}) = 0 \qquad \text{(B-1)}$$

TABLE B-1

PHYSICAL QUANTITIES CONSIDERED FOR THE RIGID BEARING PLATE TESTS OF FLEXIBLE PAVEMENTS

PHYSICAL QUANTITY	SYMBOL	FUNDAMENTAL UNITS
Deflection	x	L
Applied pressure	p	FL^{-2}
Thickness of asphaltic concrete	a	L
Thickness of base course	b	L
Bearing plate diameter	d	L
Time	t	T
Soil unconfined compressive strength	q	FL^{-2}
Characteristic soil viscosity	η	$FL^{-2}T$
Asphaltic concrete strength parameter	k_1	FL^{-2}
Characteristic asphaltic concrete viscosity	C_1	$FL^{-2}T$
Base course strength parameter	k_2	FL^{-2}
Characteristic base course viscosity	C_2	$FL^{-2}T$
Number of load applications	N	$F^0L^0T^0$

The physical quantities given in Table B-1 have been selected for use in the dimensional analysis of the problem of the rigid plate bearing test on the surface of a flexible pavement section. A force-length-time system of fundamental units has been employed.

It is assumed that the material constants needed to describe the deformation characteristics of the subgrade and various pavement layers are implicit in characteristic strength and viscosity parameters for the materials. The characteristic strength parameters may be unconfined or triaxial compressive strengths, compression or shear moduli, or some other form of material strength property, such as density. The characteristic viscosity parameter for a material controls the rate at which the deformation takes place and may include non-Newtonian effects. This is an important factor about which very little applicable quantitative information is known in the case of many engineering materials. For circular bearing plates, the effect of geometry is expressed by the diameter.

Because there are thirteen physical quantities and three fundamental units, there must be ten independent nondimensional π terms. By a methodical process described by Kondner (*32*), the following terms can be obtained:

$$\pi_1 = \frac{p}{q} \qquad \pi_6 = \frac{k_2}{q}$$

$$\pi_2 = \frac{x}{d} \qquad \pi_7 = \frac{qt}{\eta}$$

$$\pi_3 = \frac{a}{d} \qquad \pi_8 = \frac{k_1 t}{C_1}$$

$$\pi_4 = \frac{b}{d} \qquad \pi_9 = \frac{k_2 t}{C_2}$$

$$\pi_5 = \frac{k_1}{q} \qquad \pi_{10} = N$$

These terms can be substituted in Eq. B-1 to obtain the function F.

A general interpretation of these π terms can be given as follows. The terms π_1, π_5, and π_6 express the strength ratios of the subgrade, asphaltic concrete, and base course, respectively. The ratios of the time of loading to the relaxation time for the subgrade, asphaltic concrete, and base course are given by π_7, π_8, and π_9, respectively. The terms given by π_3 and π_4 are characteristic length ratios, while π_2 is the settlement parameter. The number of load applications is dimensionless and is given by π_{10}. Considering p/q as the dependent variable for the study, the functional relationship given by Eq. B-1 can be written

$$\frac{p}{q} = F\left[\frac{x}{d}, \frac{a}{d}, \frac{b}{d}, \frac{k_1}{q}, \frac{k_2}{q}, \frac{qt}{\eta}, \frac{k_1 t}{C_1}, \frac{k_2 t}{C_2}, N\right] \qquad \text{(B-2)}$$

where F denotes "some function of," but not necessarily the same function in every instance.

For the reported Hybla Valley test data, q, η, k_1, C_1, k_2, and C_2 are assumed constant; hence, if t can be regarded as sufficiently large that its effect is minimized, the terms k_1/q, k_2/q, $q\,t/\eta$, $k_1\,t/C_1$, and $k_2 t/C_2$ may be considered nearly constant and eliminated from Eq. B-2. This does not mean that the load-deflection response is independent of the type and quality of pavement materials, but only that the pavement materials were constant for the data analyzed. It is to be expected that the relationships would be different for different pavement materials, and perhaps even for different types of loading. Thus, restricting the analysis to the Hybla Valley data and the assumptions mentioned, Eq. B-2 may be written

$$\frac{p}{q} = F\left[\frac{x}{d}, \frac{a}{d}, \frac{b}{d}\ N\right] \qquad (30)$$

For the portions of the analysis which deal with only one load application, Eq. 30 may be further reduced to

$$\frac{p}{q} = F\left[\frac{x}{d}, \frac{a}{d}, \frac{b}{d}\right] \qquad (31)$$

According to Yoder (*33*), the classical linear elastic analysis of flexible pavement deflections yields equations of the form

$$\frac{x}{d} = \frac{p}{E}K \qquad \text{(B-3)}$$

where E is the modulus of elasticity of the subgrade and K is a dimensionless factor which is a function of the thicknesses and moduli of the pavement layers. Perhaps the oldest method of analysis is based on the Boussinesq stress problem, which deals with an ideally elastic, homogeneous, isotropic, weightless half-space. This analysis was later modified by Burmister (*34, 35*) to account for layered systems. Fox (*36*) extended the Burmister solution, and Jones (*37*) presented tabulated stresses for the three-layer problem. The combination of Eq. B-3 with a stiffness relation was first accomplished by Palmer and Barber (*38*) and later expanded by Baker and Papazian (*39*). Researchers are constantly attempting to check theoretical calculations with actual values existing in flexible pavement systems, but as a rule such quantitative checks are not obtained. One reason may be the inherent linearity between load and deflection implied by Eq. B-3. The more general relation expressed by Eq. 31 does not include this restriction of linearity, and such an assumption is proposed in

this analysis. The particular analytic form of Eq. 31 for the Hybla Valley test results is determined by an empirical investigation of the actual data.

EXPERIMENTAL RESULTS

Previous Studies of Hybla Valley Data

At least two previous attempts have been made to analyze various portions of the Hybla Valley data reported in HRB SR 46. The first of these was reported by Ingimarsson (*5*) wherein he analyzed 89 rigid plate bearing tests on 26 different flexible pavement sections; these data appear in Tables 4 and 7 of HRB SR 46 (*4*). Using the linear perimeter-shear equation developed by Housel (*40*), Ingimarsson carries his analysis to the point of determining the stress reactions developed by the flexible surfaces and supporting subgrade and presents his results graphically. Ingimarsson considered the initial repetitive loading cycle of the accelerated test procedure as a seating process and concentrated his analysis on the higher ranges of load aimed at determining the ultimate supporting capacity of the flexible pavements. Although several strong trends are consistent throughout the entire test series, Ingimarsson found the complete interpretation of these stress reactions peculiarly complex. In a discussion of Ingimarsson's work, Housel (7) raises the following points which are yet to be answered. The limit of the supporting capacity for the Hybla Valley tests is not reached until the deflection is much higher than the range of thousandths of an inch normally considered in current practice. Determining the source of these abnormally high deflections and correspondingly high values of perimeter shear is peculiarly perplexing. Housel further suggests that much of the difficulty with the analysis of rigid plate bearing tests on flexible pavements may be their relative rigidity and the secondary dimensional effects which they induce; these effects may mask the basic supporting capacity which the tests attempt to measure. After spending some time in an attempt to interpret the stress reactions developed in the Hybla Valley tests in the quantitative terms of the linear equation for bearing capacity used by Ingimarsson, Housel has not come to any final conclusion.

A second attempt to analyze portions of the Hybla Valley data was reported by Kondner and Krizek (*6*). Their analysis considered only data from Table 4 of HRB SR 46 (*4*) and was based on methods of dimensional analysis. In contrast to the range of deflection considered by Ingimarsson, these authors examine the low deflection region of the data. They approximate the load-deflection relationship by a straight line through the origin and explicitly express the surface deflection as a function of the applied load, bearing plate diameter, various pavement thicknesses, and strength characteristics of the subgrade. Although there are various inconsistencies among the data which prohibit the deduction of any final quantitative conclusions, one of the major contributions of this work is the indication that both model and prototype research investigations designed, conducted and analyzed on the basis of nondimensional techniques may prevent unnecessary duplication of costly, time-consuming experimental work.

General Approach

The analysis pursued herein is based on the methods of dimensional analysis as explained earlier in this report and is a continuation of the approach previously reported by Kondner and Krizek (*6*). However, three variations are included in an effort to make the analysis more general. First, the data analyzed are more extensive, including rigid plates on the subgrade alone (*41*, Table 3), subgrade plus asphaltic concrete surface course (*41*, Table 7), subgrade plus base course and surface course (*41*, Tables 4 and 8), temperature variations (*41*, Table 6) and repetitional loading (*41*, Table 8). Second, consideration is given to the deflections of the various component layers of the pavement structure. The significance of such layer deflections has been reported by Walker et al. (*27*). Third, the load-deflection relationship is approximated by a hyperbola in rectangular Cartesian coordinates. It is felt that this equation (rather than the simple proportionality afforded by a linear one) makes possible a better fit of available data and facilitates a better correlation among the various pavement component thicknesses.

The properties of a hyperbola lend themselves quite appropriately to the response observed in a typical load-deflection test. Virtually all load-deflection plots of plate bearing tests on flexible pavements or soil begin with some initial slope and exhibit a gradual curvature until the ultimate supporting capacity is reached. On the other hand, the equation for a two-constant hyperbola in rectangular Cartesian coordinates can be written

$$y = \frac{x}{A + B\,x} \tag{B-4}$$

and demonstrates the same general behavior. Differentiation of Eq. B-4 with respect to the independent variable, x, and evaluation as x vanishes gives the initial slope of the y versus x plot as $1/A$; also, as x becomes very large the dependent variable, y, approaches the limit $1/B$. These parameters are shown graphically in Figure B-1 for a typical hyperbola. The constants A and B in Eq. B-4 are quite easy to obtain empirically. Equation B-4 may be rearranged in terms of transformed variables as

$$\frac{x}{y} = A + B\,x \tag{B-5}$$

Thus, if the transformed variables x/y and x are plotted versus each other, as shown in Figure B-2, the constants A and B are simply the intercept and slope, respectively, of the straight line approximating the data. It is not to be expected that all experimental data will lie exactly on such a straight line nor is it to be implied that a hyperbola is the optimum equation to describe the data in all cases. The hyperbola has been selected simply because it appears to offer a very reasonable analytic description for a large variety of experimental data dealing with load-deformation response. Several useful applications of hyperbolic fits have been reported by Kondner (*42*) for the analysis of friction pile behavior, Kondner (*43*) for the analysis of stress-strain response, and Kondner and Krizek (*31*) for the correlation of load bearing tests.

The original data corresponding to each individual test are examined in terms of the dimensionless parameters p/q

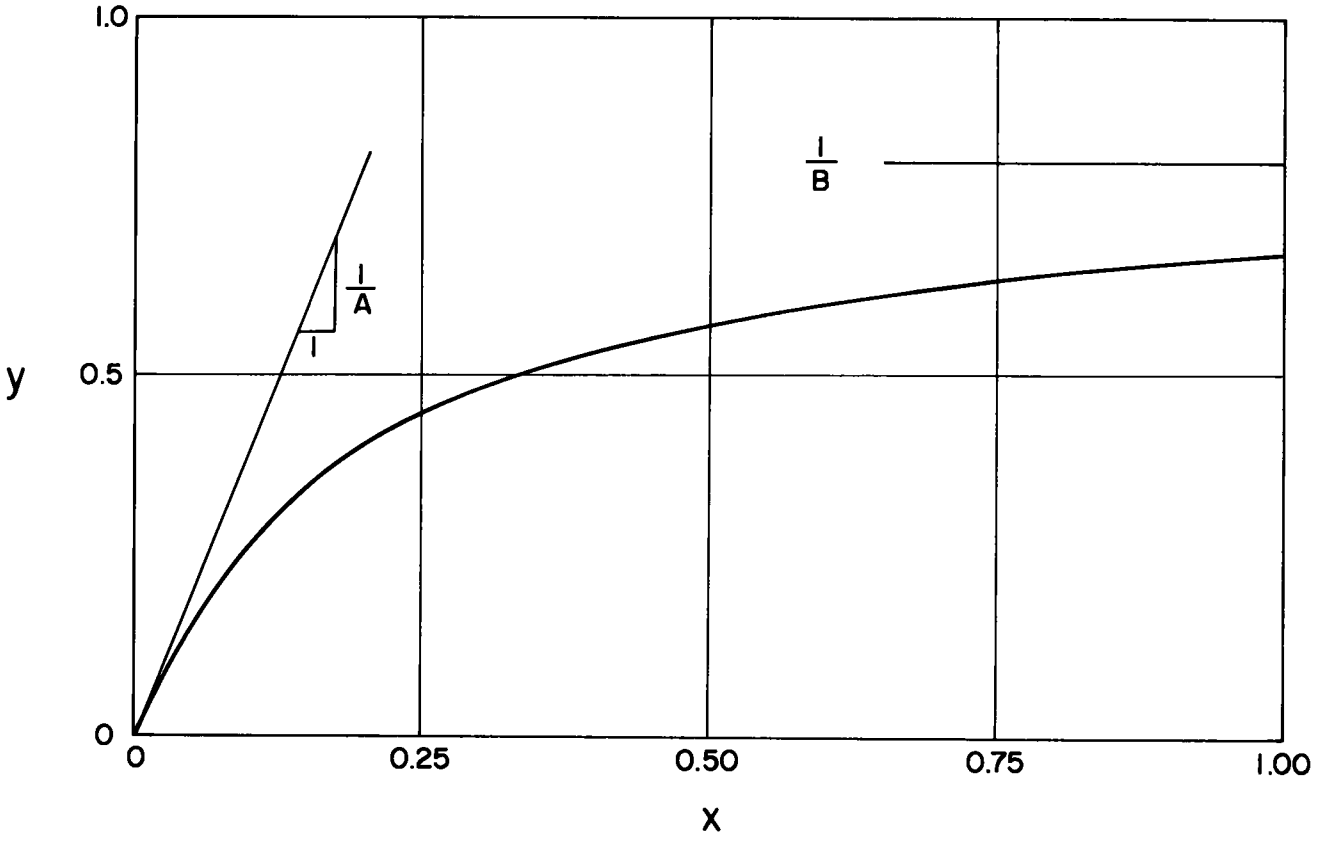

Figure B-1. Typical hyperbola in rectangular Cartesian coordinates.

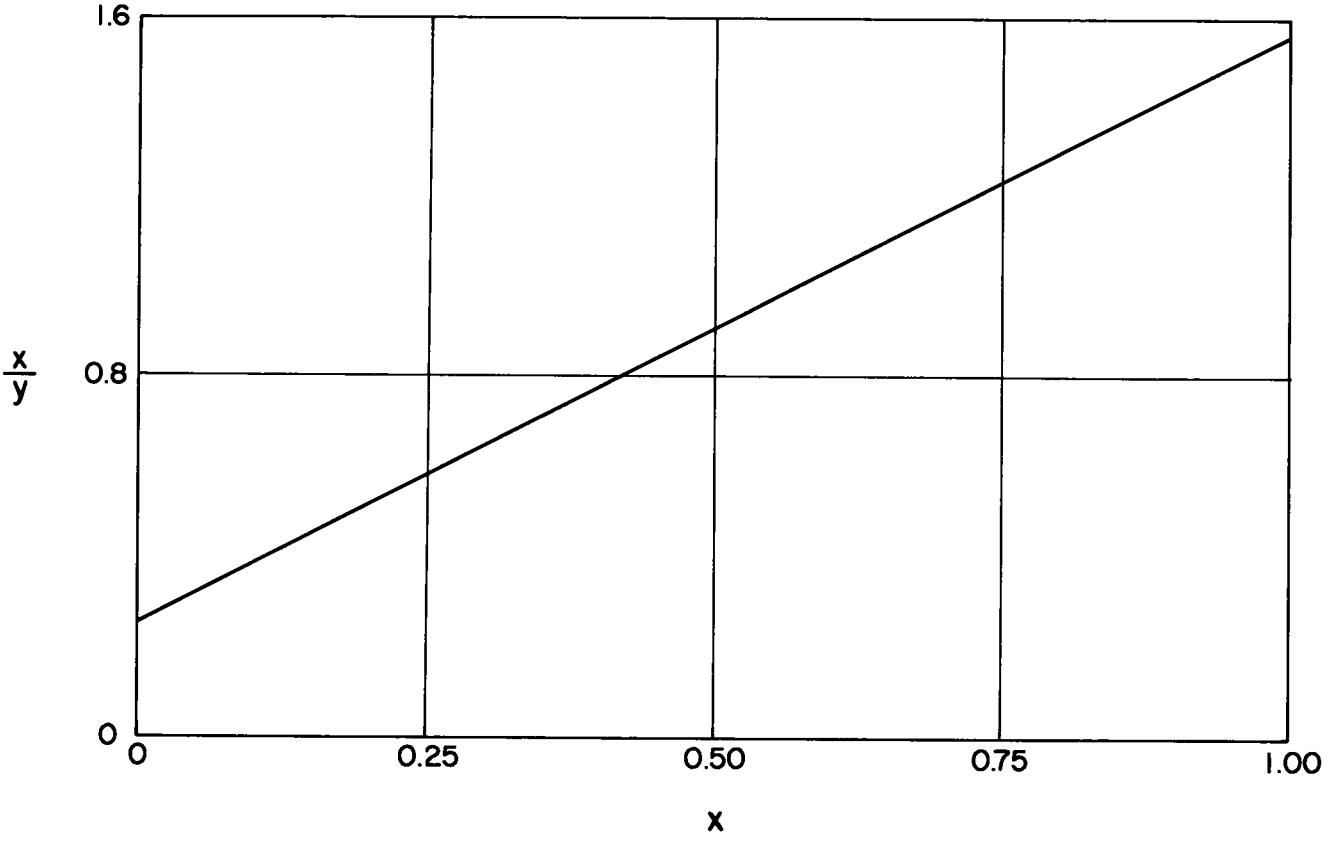

Figure B-2. Typical hyperbola in transformed coordinates.

and x/d or some modification thereof. The unconfined compressive strength, q, of the soil immediately under the test plate was apparently not measured (at least, these data are not given in HRB SR 46), nor was any measure of water content taken from which q might be estimated. In view of this, and because most of the tests were conducted in a relatively short period of time, it might be reasonable to expect that the soil strength varied little during the test period; therefore, for purposes of this analysis it is considered constant. Furthermore, because an actual value is not readily available, results are expressed in terms of a normalized unconfined compressive strength, q_N, equal to unity. If the required soil strength data become available, modification of the developed equations may be made to eliminate q_N and express the equations in terms of q.

In transformed hyperbolic terms the parameters to be plotted become $\frac{x\,q}{d\,p}$ and $\frac{x}{d}$, and the equation may be written, in terms of the original dimensionless parameters p/q_N and x/d, as

$$\frac{p}{q_N} = \frac{x/d}{A + B\ x/d} \tag{32}$$

where the variable, x, may represent gross deflection x_G, pavement deflection x_P, base course deflection x_B, or subgrade deflection x_S. The specific deflection under study at any point is identified appropriately, although the form of Eq. 32 is used to express all response data. If desired, the dependent and independent variables in Eq. 32 may be interchanged, and the load-deflection response can be expressed by

$$\frac{x}{d} = \frac{A\ p/q_N}{1 - B\ p/q_N} \tag{B-6}$$

The graph of these transformed hyperbolic variables is approximated by a "best fit" straight line as determined by observation, and the resulting dimensionless coefficients A and B are then examined to deduce their dependence on thicknesses of component pavement layers and number of repetitions.

Duplicability of Test Results

To possess a high degree of reliability, any test program must exhibit good duplicability for any test within its scope. Such a condition is not satisfied in many cases for the Hybla Valley test results. One example of good duplicability for applied surface pressure, p, versus pavement deflection x_P, base course deflection x_B, and subgrade deflection, x_S, is shown in Figures B-3, B-4, and B-5. On the other hand, Figure B-6 shows a typical example of poor duplicability. For data such as shown in Figure B-6, a coherent analysis is not possible unless some estimate is made regarding the test most nearly representing the "true" situation. Most of the time in the analysis herein an average value was used; however, occasionally one or two tests were disregarded completely and the average of the remaining tests was used.

Figures B-7 through B-11 show several examples of test data plotted in transformed hyperbolic parameters. These figures will illustrate the difficulty encountered on some occasions in the determination of the dimensionless constants A and B. Figure B-7 shows very good duplicability and a good hyperbolic fit; hence, A and B can be determined within quite small limits. Figure B-8 shows the results of two similar tests which exhibit essentially the same slope, B, but quite a different intercept, A. The choice becomes somewhat subjective. Figure B-9 gives the results of two supposedly similar tests which demonstrate drastically divergent behavior. The negative slope indicates that the curvature of the load-deflection data is opposite to that shown in Figure B-1, or that the resistance to deformation becomes greater as the load increases. Figure B-10 provides a good example of two tests where the average results provide a quite acceptable straight line, but where the individual tests themselves are open to substantial doubt. Finally, Figure B-11 shows the results of four supposedly identical tests, each of which produces a different slope and intercept. This is one example of a case where the uppermost test was disregarded and the average of the remaining tests was used to determine an acceptable straight line to represent the data.

It is evident that such divergent results can never be reconciled in any coherent exact analysis. Furthermore, in the absence of any explanation for such variance in response among supposedly identical tests, the results of other tests in the program must be viewed with suspicion. Thus, in this light it appears that the establishment of trends among the variables must be an end in itself and any attempt at an exact analysis encompassing all test data must be precluded.

Test Data on Subgrade

This section considers the analysis of data resulting from rigid bearing plates resting directly on the subgrade. Some aspects of these data were previously considered by Kondner and Krizek (*41*). The data are tabulated in Table 3 of HRB SR 46 (*4*) and tests were conducted in the following manner. Bearing plates ranging from 12 to 84 in. in diameter were loaded to a specified level and the gross deflection was measured; following this, the load was released and the elastic deflection was measured. This process was repeated four more times at the same load level. Then the load was increased to three successively higher levels, there being five cycles with corresponding measurements at each particular level. The data used herein result from the gross deflections for the first cycle at each load level for the 17 tests conducted on the subgrade of the south tangent. It is recognized that measurements made on the second, third, and fourth series may be influenced by the history imposed by previous loadings, but in the absence of any information to quantitatively evaluate this effect all data from each series are considered together. These data are plotted in terms of the transformed hyperbolic parameters $\frac{x_G\,q_N}{d\,p}$ and $\frac{x_G}{d}$ in Figure B-12. Approximating these data by the straight line shown, the dimensionless coefficients A and B are found to be 0.00018 and 0.0125, respectively. Hence, following the general form of Eq. 32, a descriptive equation for these data may be written as

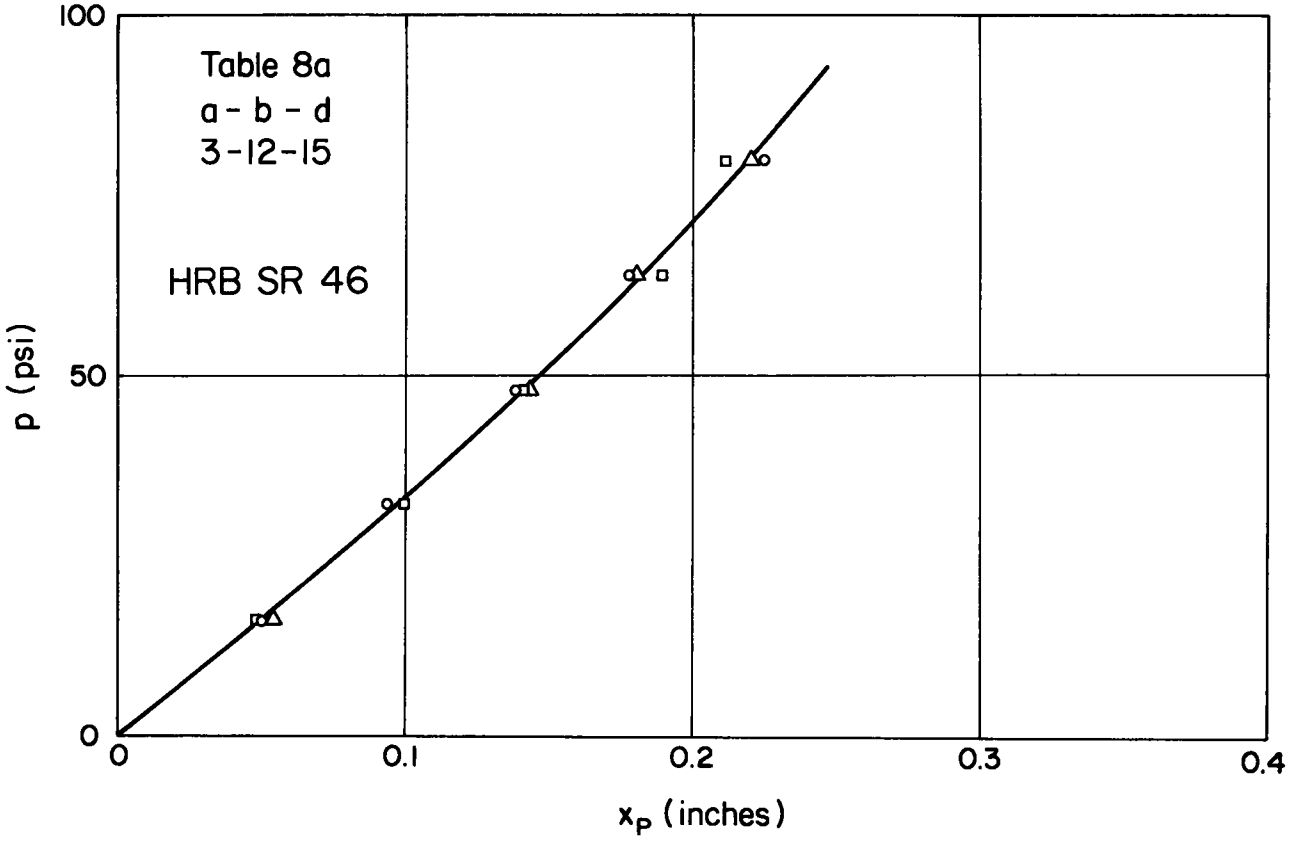

Figure B-3. Applied pressure vs pavement deflection, showing good duplicability.

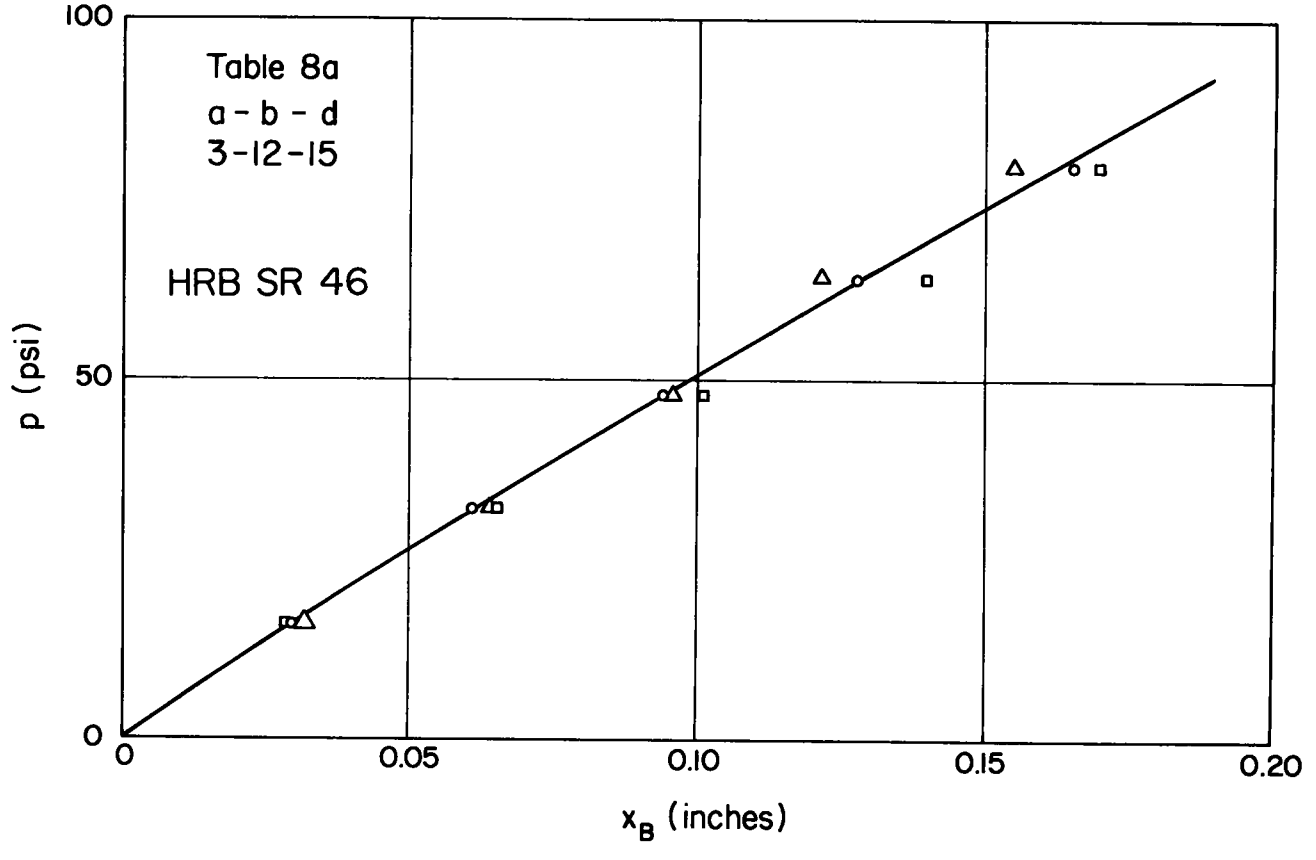

Figure B-4. Applied pressure vs base course deflection, showing good duplicability.

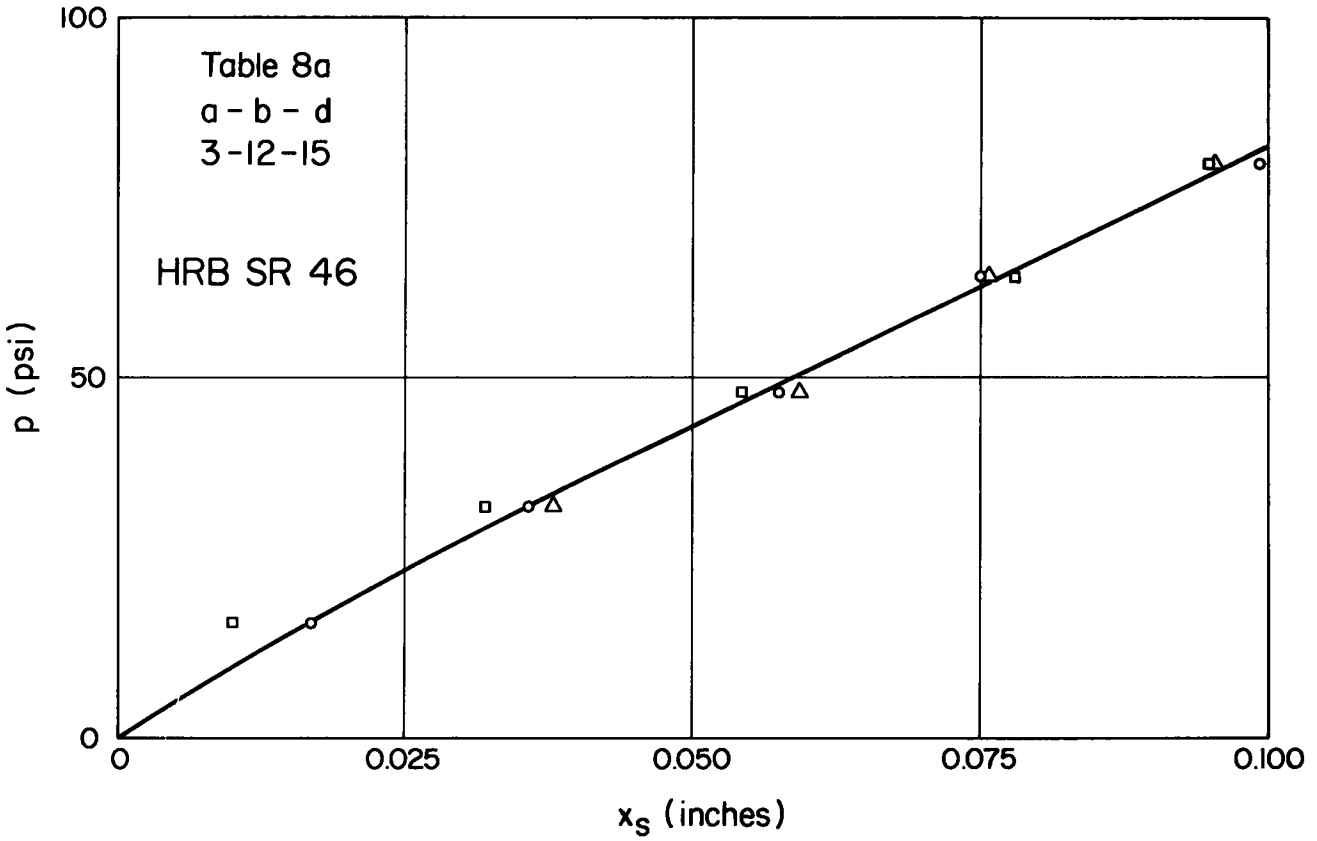

Figure B-5. Applied pressure vs subgrade deflection, showing good duplicability.

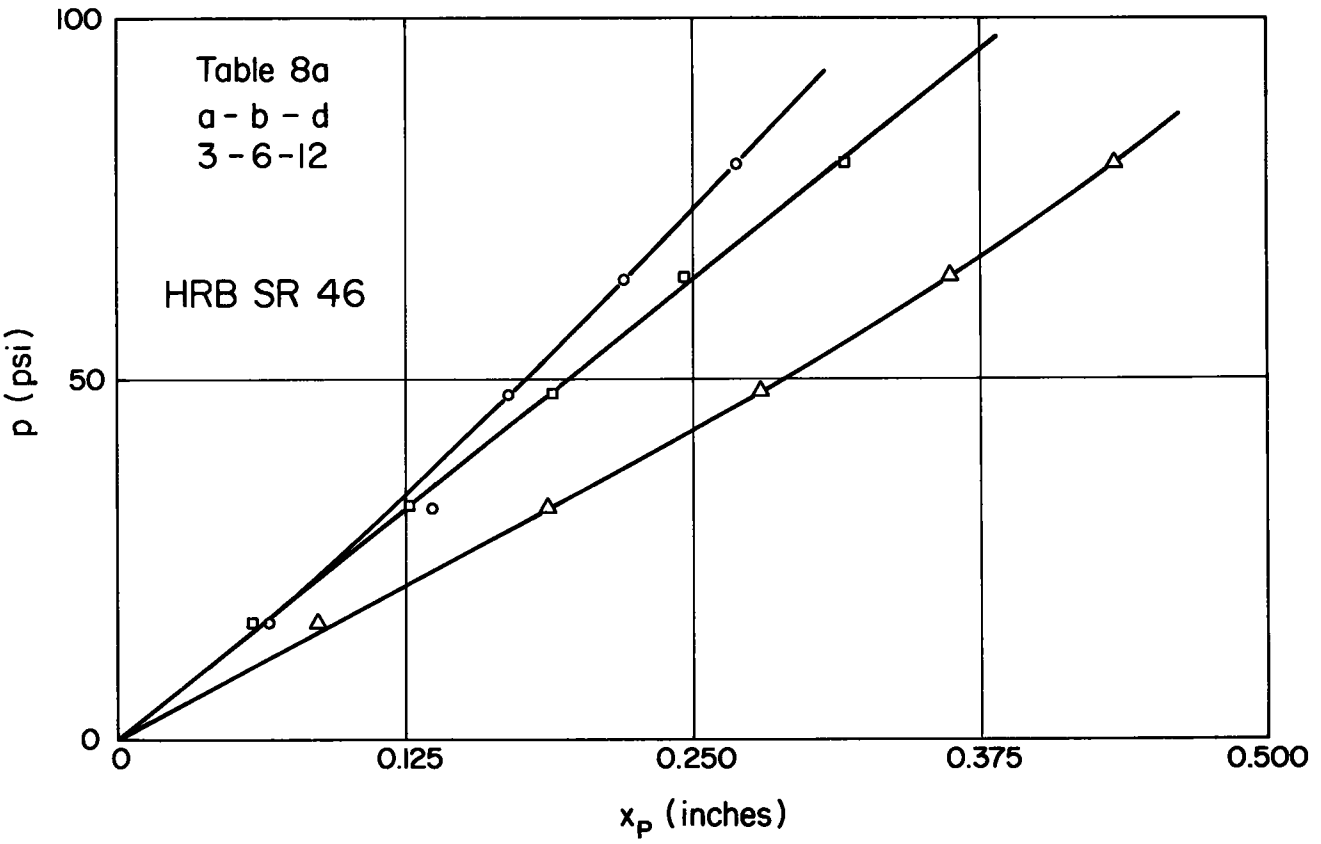

Figure B-6. Applied pressure vs pavement deflection, showing poor duplicability.

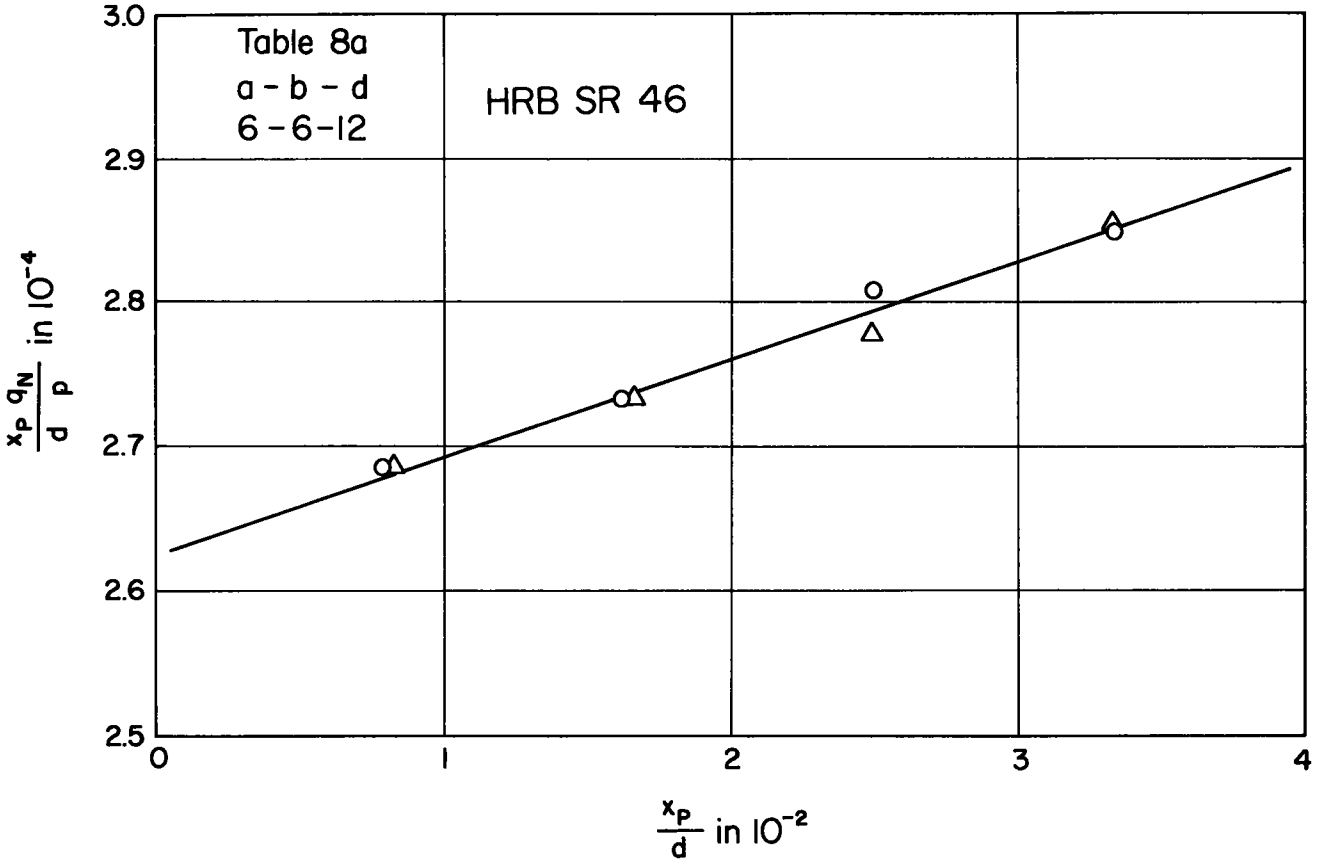

Figure B-7. Transformed plot of applied pressure parameter vs pavement deflection parameter, showing good duplicability.

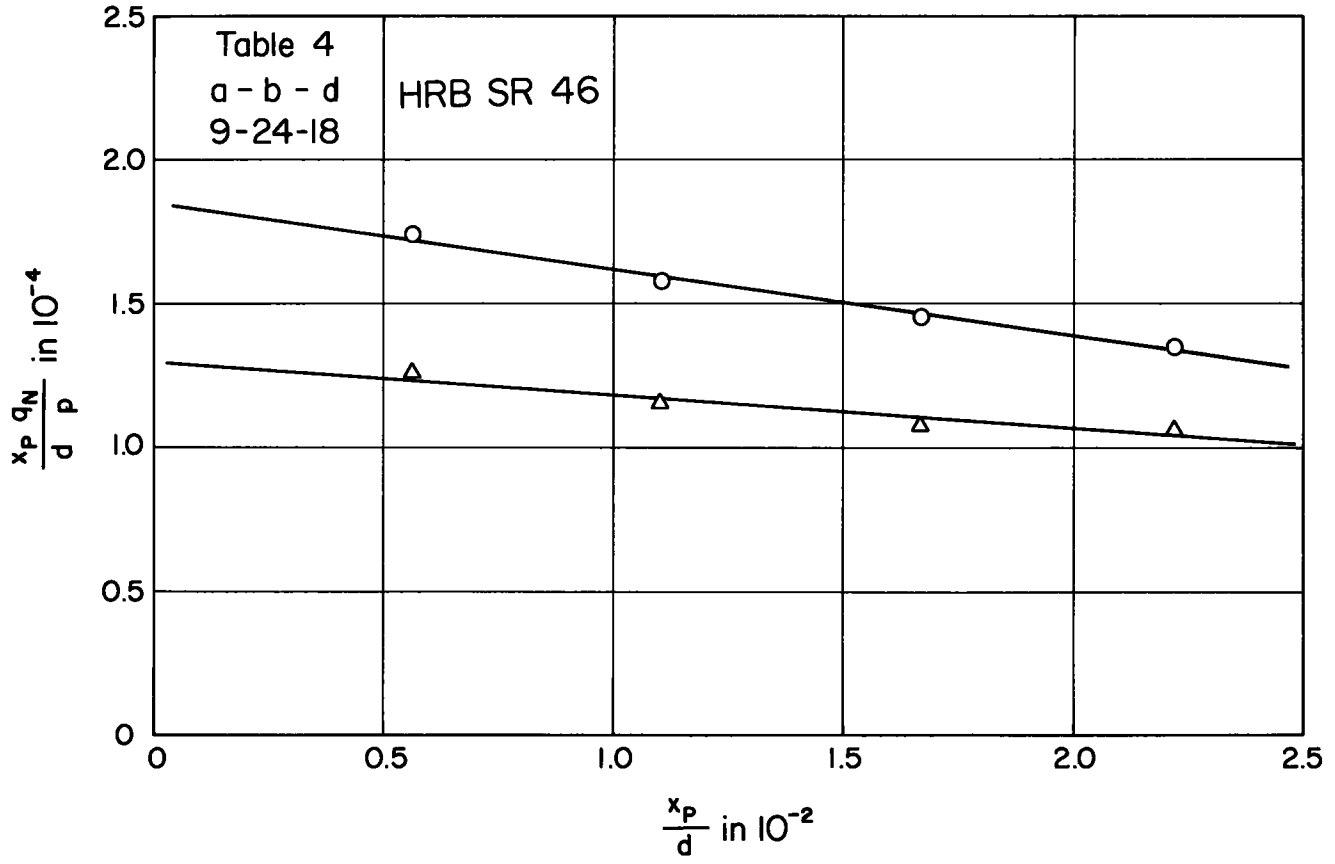

Figure B-8. Transformed plot of applied pressure parameter vs pavement deflection parameter, showing poor duplicability.

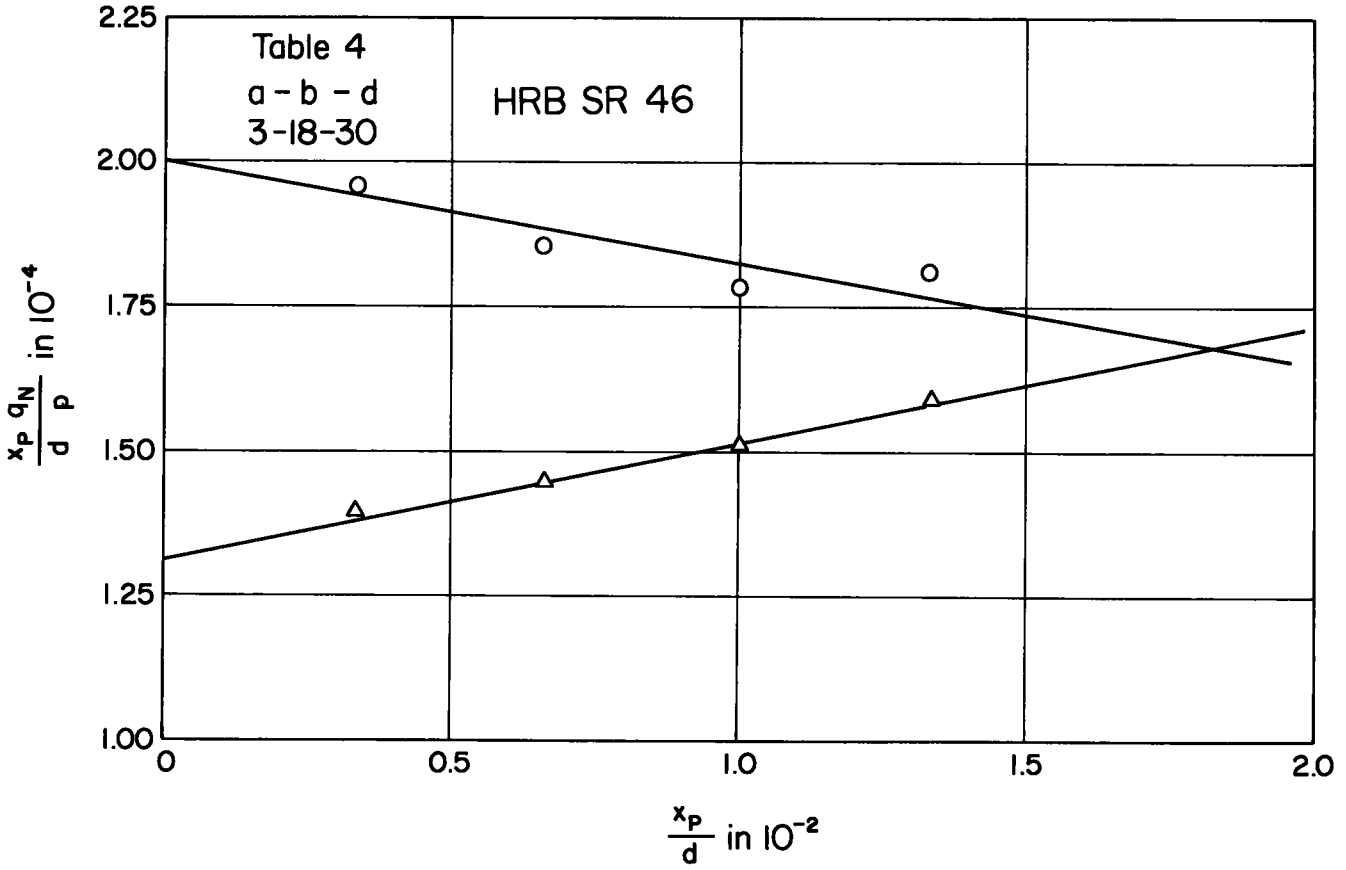

Figure B-9. Transformed plot of applied pressure parameter vs pavement deflection parameter, showing poor duplicability.

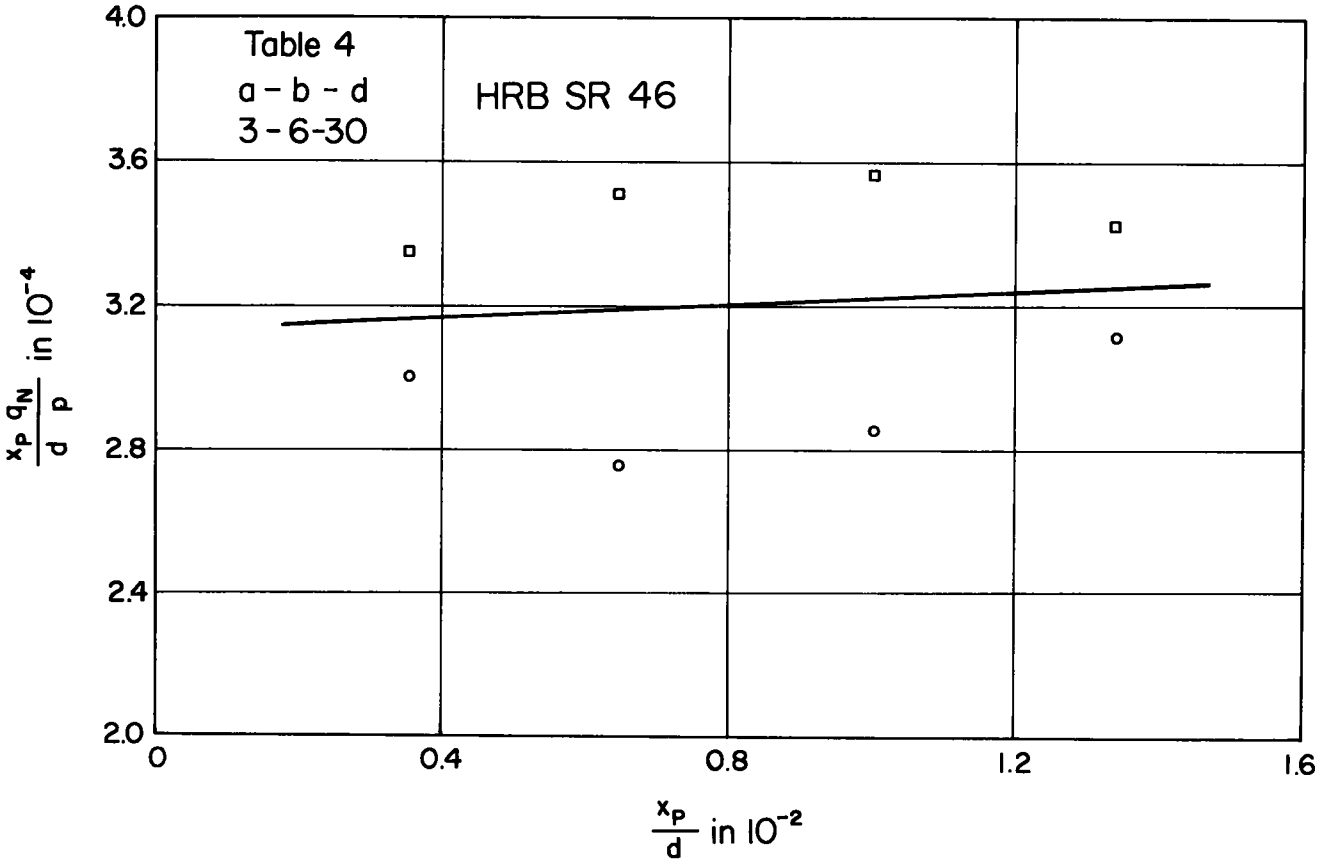

Figure B-10. Transformed plot of applied pressure parameter vs pavement deflection parameter, showing poor duplicability.

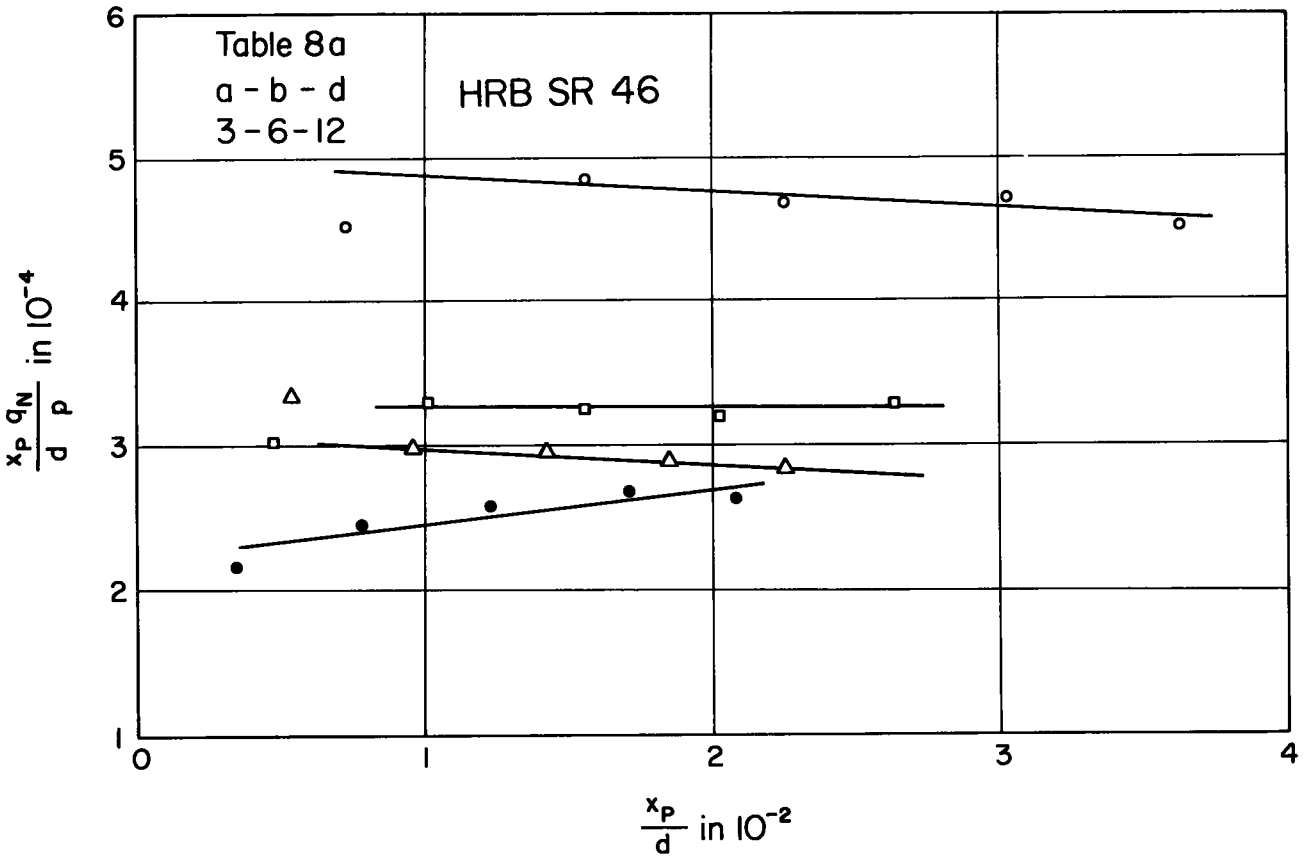

Figure B-11. Transformed plot of applied pressure parameter vs pavement deflection parameter, showing poor duplicability.

$$\frac{p}{q} = \frac{x_G/d}{0.00018 + 0.0125\, x_G/d} \tag{33}$$

Test Data on Subgrade Plus Surface Course

The data used in this section were obtained from Table 7 of HRB SR 46 (*4*). For these tests the subgrade was overlain by an asphaltic surface course with a thickness of 3, 6 or 12 in. Rigid bearing plates with diameters of 12, 18, 24, and 30 in. were employed. The test data were obtained by the accelerated test procedure, which is described in HRB SR 46 (*4*). In brief, this test procedure consists of measuring the load required to cause a surface deflection of 0.1 in., then releasing the load; the same is done for surface deflections of 0.2, 0.3, and 0.4 in. After release of the load causing a 0.4-in. surface deflection, a continuous load of increasing magnitude is applied until (a) the material is unable to support a further increase, or (b) the gross deflection exceeds 2 in., or (c) the total reaction is utilized. The rate of application of the load is controlled so as to produce a rate of vertical movement of the surface under test of 0.5 in./min. The magnitudes of the loads to produce 0.1-, 0.2-, 0.3-, and 0.4-in. surface deflection are estimated and the respective deflections are intended to be guides only; however, in the absence of the actual deflections (which are not recorded in Table 7) the only recourse is to use these guides as the "true" deflections. The magnitude of the error so induced can not be readily determined.

Utilizing the data for the initial portion of each test (that is, up to deflections of 0.4 in.), the results are plotted in terms of transformed hyperbolic parameters and values for A and B are determined. In general, these values may be regarded as functions of the dimensionless term a/d. A plot of A versus a/d is shown in Figure B-13. These data can be approximated by the straight line

$$A = 0.00018 + 0.00012\, a/d \tag{34}$$

The graph of B versus a/d for these data is given in Figure B-14; no descriptive analytic equation is immediately observable. However, if these data are replotted in the form of $(a/d)/(0.0125 - B)$ versus a/d, as shown in Figure B-15, a reasonably descriptive equation may be written as

$$\frac{a/d}{0.0125 - B} = 12 + 70\, a/d \tag{B-7}$$

Solving Eq. B-7 for B yields

$$B = 0.0125 - \frac{a/d}{12 + 70\, a/d} \tag{35}$$

It should be noted that when a/d reduces to zero (that is, the plate rests directly on the subgrade), Eqs. 34 and 35 reduce to the values for A and B as determined in the previous section for the analysis of bearing plates on subgrade. Also that in Figures B-13 and B-14 values for A

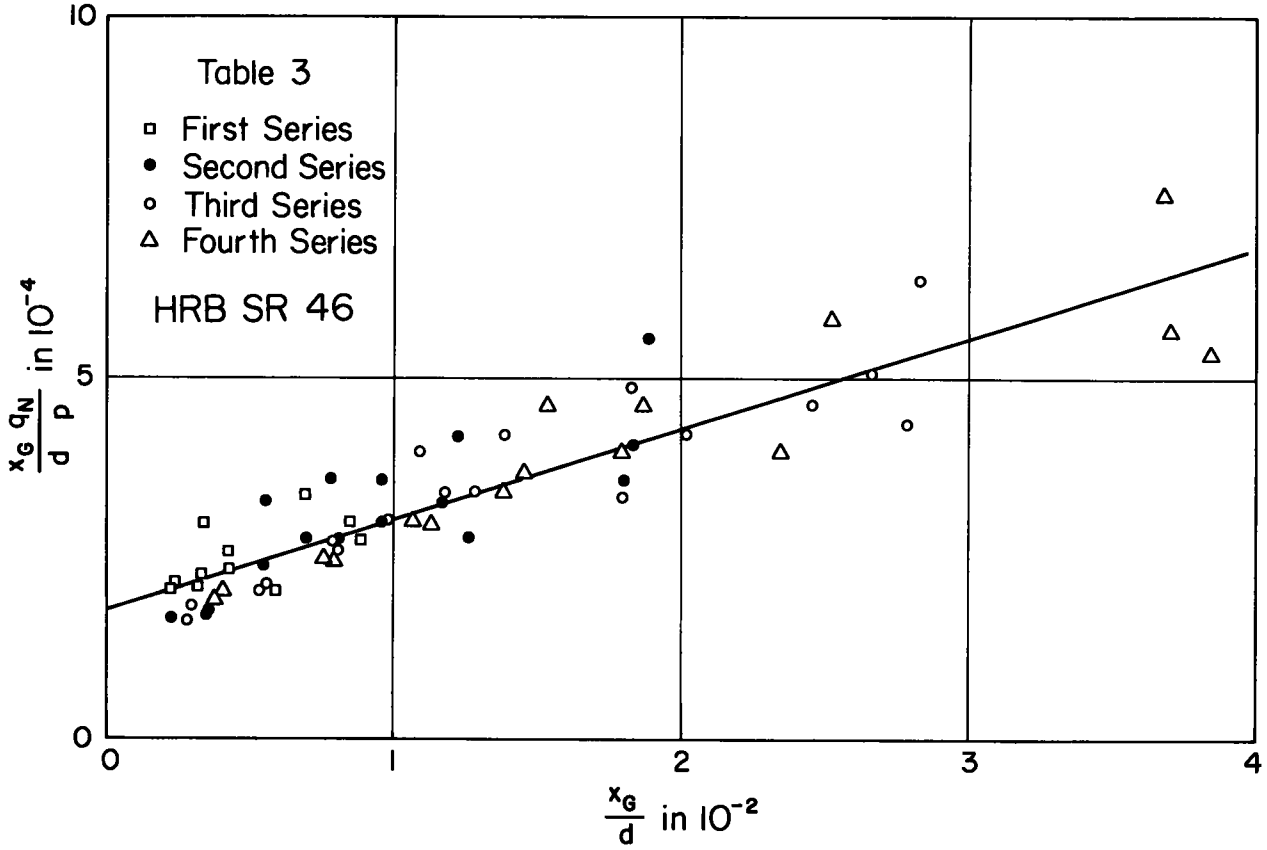

Figure B-12. Transformed plot of applied pressure parameter vs gross subgrade deflection parameter.

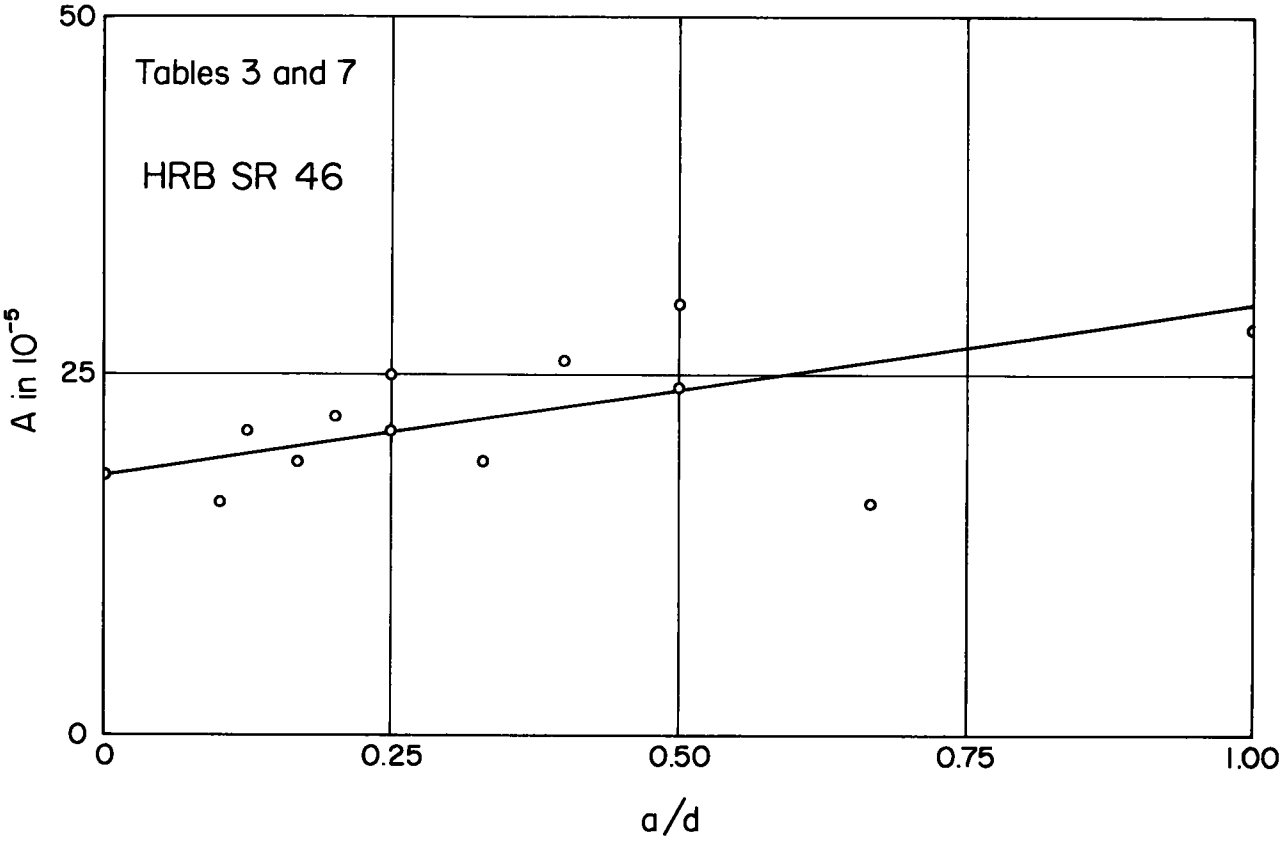

Figure B-13. Intercept parameter vs surface course thickness parameter.

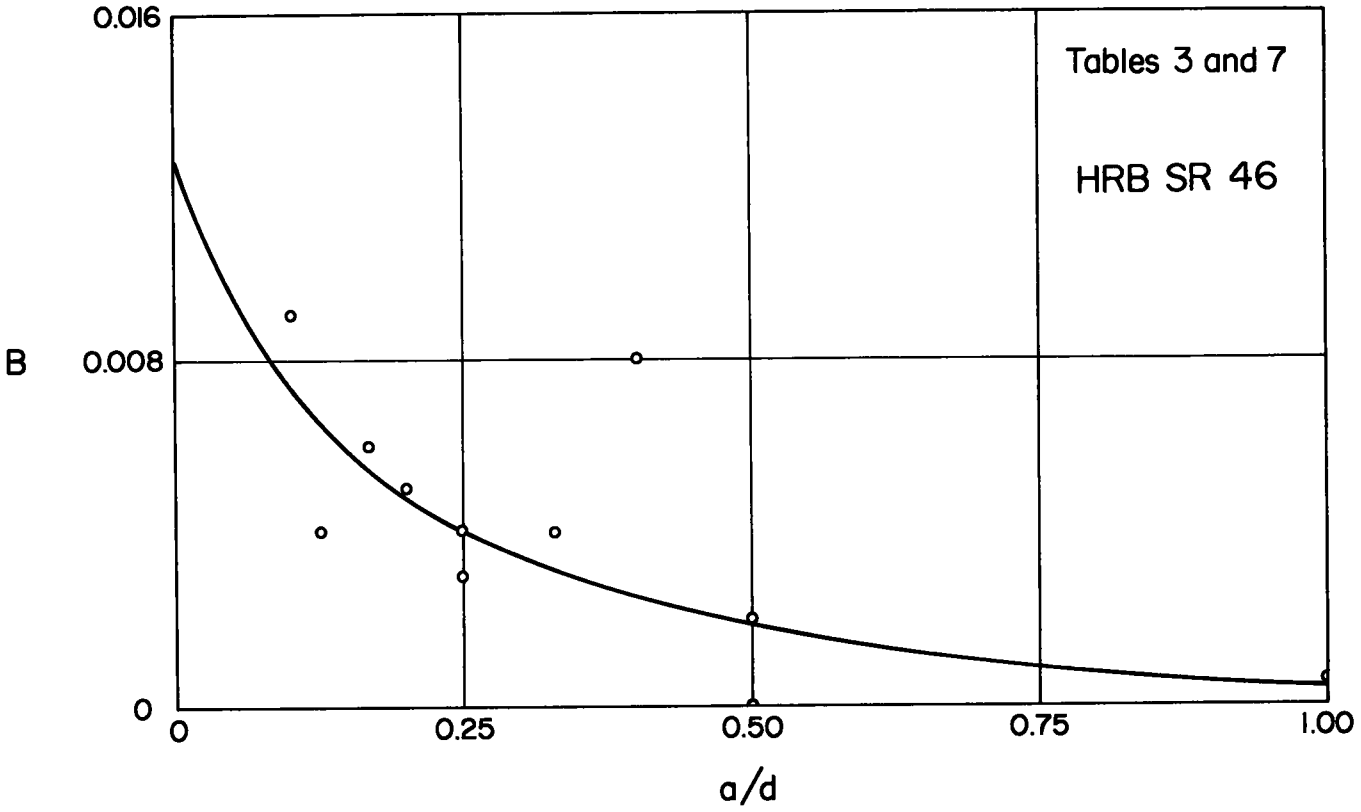

Figure B-14. Slope parameter vs surface course thickness parameter.

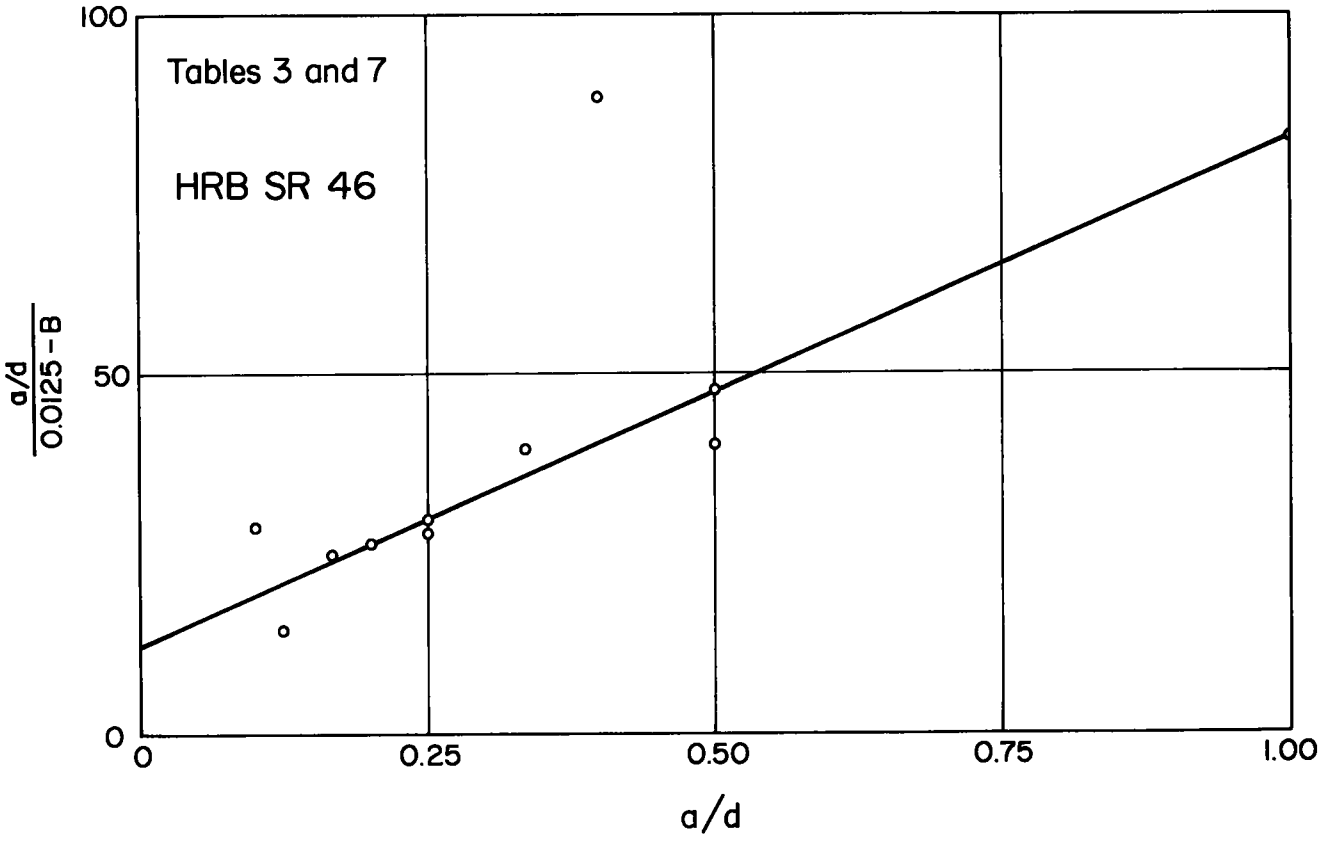

Figure B-15. Transformed plot of slope parameter vs surface course thickness parameter.

increase slightly with increasing a/d, whereas values for B decrease considerably with increasing a/d. This means that for a given diameter plate the presence of a thicker asphaltic concrete surface course causes the initial slope of the load-deflection graph to be somewhat flatter (less stiff) while the ultimate supporting capacity is significantly increased. Substituting Eqs. 34 and 35 in Eq. 32 gives the general equation representing the trend of response for rigid bearing plates on subgrade or subgrade overlain with an asphaltic surface course as

$$\frac{p}{q} = \frac{x_P/d}{0.00018 + 0.00012\, a/d + \left[0.0125 - \dfrac{a/d}{12 + 70\, a/d}\right] x_P/d} \tag{36}$$

Test Data on Subgrade Plus Surface and Base Courses

DATA ANALYZED AND DISCUSSION OF TECHNIQUES

The data analyzed in this section are taken from Table 4 of HRB SR 46 (*4*). They consist of the surface or pavement, base course, and subgrade deflection data from the cyclic portion of the accelerated test procedure (described in the previous section) for the complete pavement sections; that is, the pavement sections for which no component layer was removed and the bearing plate was placed on the asphaltic concrete surface layer. However, for such sections deflections were measured at all three layers by a procedure described in HRB SR 46. The significance of layer deflection measurement has been discussed by Walker et al. (*27*).

In general, the method of approach to this analysis is similar to that already described. Individual test data are plotted in transformed hyperbolic form and the results are approximated by a straight line from which the dimensionless coefficients A and B can be determined. These coefficients are then investigated as functions of the characteristic layer thickness parameters a/d and b/d, or some reasonable combination thereof.

Because the data in the previous section represent those obtained for the case where $b/d = 0$, the first attempt was to plot values for A and B versus a/d (similar to Figs. B-13 and B-14) for constant values of b/d. However, no trends could be established using this approach. Likewise, plotting A and B versus b/d for constant values of a/d produced no conclusive trends. Various unsuccessful attempts were made to establish some trend while maintaining the independence of a/d and b/d. A significant effort was directed toward the formulation of an equation among these variables which would yield reasonable results in the limit; in other words, it was desired to describe these data in Table 4 of HRB SR 46 by a generalized analytic expression which would degenerate to Eq. 36 when b/d vanishes and to Eq. 33 when both a/d and b/d vanish. After several unsuccessful attempts, it was decided to pursue the task of establishing trends which are limited in their applicability to the ranges of variables included in Table 4 of HRB SR 46. In his paper on the methods of analysis and interpretation of flexible pavement deflections, Yoder (*33*) made the statement: "Generalization, however, can at times complicate the picture rather than simplify it." Following the failure at generalization mentioned previously, the wisdom of this statement can be duly appreciated.

The technique which was found most successful in the analysis of these data was to employ an assumed relationship between a/d and b/d. The relationship selected was that determined by the AASHO Road Test data and given in Eq. 2. The test sections at the Hybla Valley site included no subbase course; hence, the thickness index parameter can be written as

$$\frac{D}{d} = 0.44\,\frac{a}{d} + 0.14\,\frac{b}{d} \tag{B-8}$$

Because, in general, the pavement materials and subgrade for the Hybla Valley project were different from those at the AASHO Road Test, there is no reason to conclude *a priori* that the thickness index relation determined on the AASHO project will be applicable to the Hybla Valley data. However, it was found to be quite effective in reducing the Hybla Valley data to a more compact form and hence seems to be very applicable. Of course, the ultimate goal of such research is to establish relationships such as these which may be employed interchangeably on different pavement projects.

The particular forms of the variables plotted in the succeeding graphs represent extensive effort in making these choices. Preliminary choices which did not yield satisfactory relationships are not shown. The criterion for a satisfactory relationship is the absence of any orderly sequence among the variables under consideration. Although the different combinations of these variables are too numerous to employ any satisfactory or comprehensive legend in the following graphs, a detailed examination of any one graph will reveal a high degree of random scatter. The width of the bands of data appears to be the result of inconsistencies among the original tests; any attempt to narrow the band resulted in the introduction of some order among the data points. Hence, the result achieved is essentially the establishment of appropriate equations representing the trends demonstrated by these data within the respective ranges of the variables considered.

PAVEMENT DEFLECTIONS

Each test utilized in Table 4 of HRB SR 46 was plotted in terms of the transformed hyperbolic variables $\frac{x_P q}{dp}$ and $\frac{x_P}{d}$ and the dimensionless coefficients A and B were determined. If the coefficient A is multiplied by the dimensionless parameters $\frac{a}{d}$ and $\frac{b}{d}$, the resultant term, $\frac{ab}{d^2} A$, may be plotted versus the thickness index parameter $\frac{D}{d}$, as computed from Eq. B-8. This graph is shown in Figure B-16, where the data are approximated by the line shown; the equation of this line is

$$\frac{a\,b}{d^2} A = 4 \times 10^{+3.33\, D/d\, -6} \tag{B-9}$$

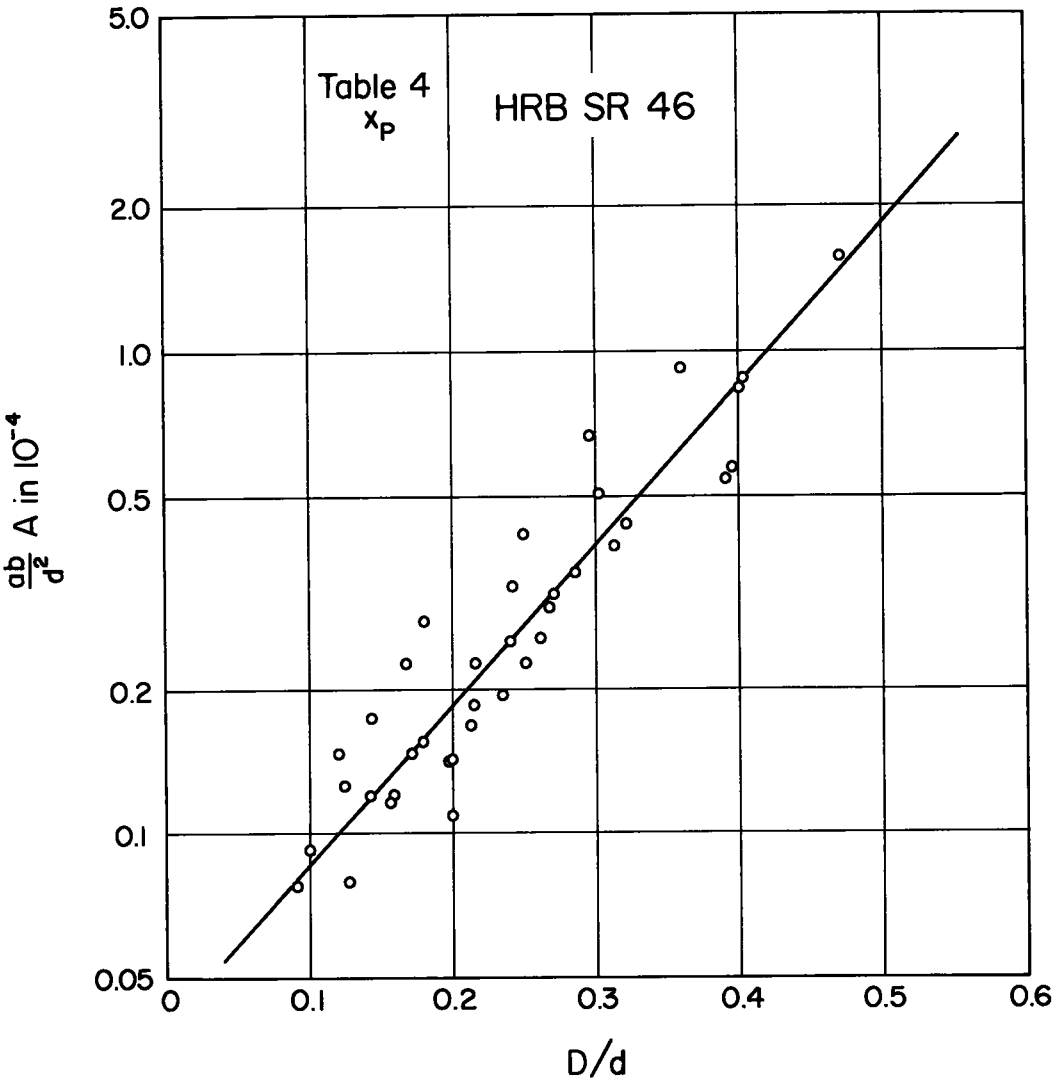

Figure B-16. Intercept parameter vs thickness index parameter, pavement deflection.

The coefficient, B, is plotted directly versus $\frac{D}{d}$ in Figure B-17; the data may be approximated by the curve shown, whose equation is

$$B = 8 \times 10^{(-3\ D/d)-3} - 0.001 \qquad (38)$$

Solving Eq. B-9 for A, and substituting this relationship plus the relationship for B given by Eq. 38 in the general hyperbolic equation (Eq. 32), the load-deflection response for the flexible pavement sections analyzed may be represented by

$$\frac{p}{q_N} = \frac{x_P/d}{\frac{4d^2}{ab} \times 10^{3.33\ D/d\ -6} + [8 \times 10^{(-3\ D/d)\ -3} - 0.001]\ x_P/d} \qquad (39)$$

BASE COURSE DEFLECTIONS

This section analyzes base course deflections, x_B, caused by a rigid bearing plate on the asphaltic concrete surface course layer. The general approach to the analysis is similar to that previously used except that the hyperbolic parameters plotted are $\frac{x_B\, q(d+2a)}{p\, d^2}$ versus $\frac{x_B}{d+2a}$. Essentially, these changes in the parameters reflect the fact that the total force applied to the bearing plate resting on the pavement surface is considered to be uniformly distributed over an area with diameter $d + 2a$ at the top of the base course layer. Of course, the presence of the asphaltic concrete surface course may also contribute a confining effect. Such a technique has been found effective in reducing the experimental data to a compact form.

The parameters mentioned were plotted for each test analyzed, and coefficients A and B were determined. These coefficients were then investigated to determine their de-

pendence on the various thicknesses of pavement components. The selected plots are shown in Figures B-18 and B-19. Figure B-18 shows the parameter $\frac{ab}{d^2}A$ plotted versus D/d, and the straight line approximating these data can be written as

$$\frac{ab}{d^2}A = 3 \times 10^{3.42\,D/d\,-6} \qquad \text{(B-10)}$$

Similarly, Figure B-19 shows $\frac{d}{a}B$ plotted versus D/d, and the equation describing this trend may be expressed

$$\frac{d}{a}B = 10^{-1.0\,-\,3.23\,D/d} \qquad \text{(B-11)}$$

Solving Eqs. B-10 and B-11 for A and B, respectively, and formulating an equation similar to Eq. 32 gives

$$\frac{p}{q_N} = \left[\frac{(d+2a)^2}{d^2}\right] \frac{x_B/(d+2a)}{\frac{3d^2}{ab} \times 10^{(3.42D/d)-6} + \left[\frac{a}{d} \times 10^{-1.0-(3.23D/d)}\right] x_B/(d+2a)} \qquad (40)$$

The trends exhibited for base course deflections, x_B, are qualitatively similar to those found for pavement deflections, x_P; of course, actual functional relationships may be somewhat in variance, as expected. Comparing the values of A for pavement and base course deflections, as may be obtained from Eqs. B-9 and B-10, respectively, it is found that the initial slope of the load-deflection plot for the base course deflection data is approximately 4/3 that for the pavement deflection data.

SUBGRADE DEFLECTIONS

Subgrade deflections, x_S, which are analyzed in this section, are handled analogous to the base course deflections in the previous section. The total force applied to the bearing plate on the surface is considered uniformly distributed over an area with diameter $d + 2a + 2b$ at the top of the subgrade. The overlying base course and surface course layers may exert a confining effect, hence the response does not necessarily follow the same form as a plate with an assumed uniform pressure resting directly on the subgrade, as analyzed in a previous section.

The dimensionless hyperbolic parameters $\frac{x_S\, q\, (d + 2a + 2b)}{p\, d^2}$ and $\frac{x_S}{d + 2a + 2b}$ are plotted for each test considered. From these plots the coefficients A and B are determined and examined to deduce their dependence on various pavement component thicknesses. Graphs involving these relationships are shown in Figures B-20 and B-21. Figure B-20 shows $\frac{d}{D}A$ plotted versus $\frac{D}{d}$, and the equation representing the trend of this response can be written

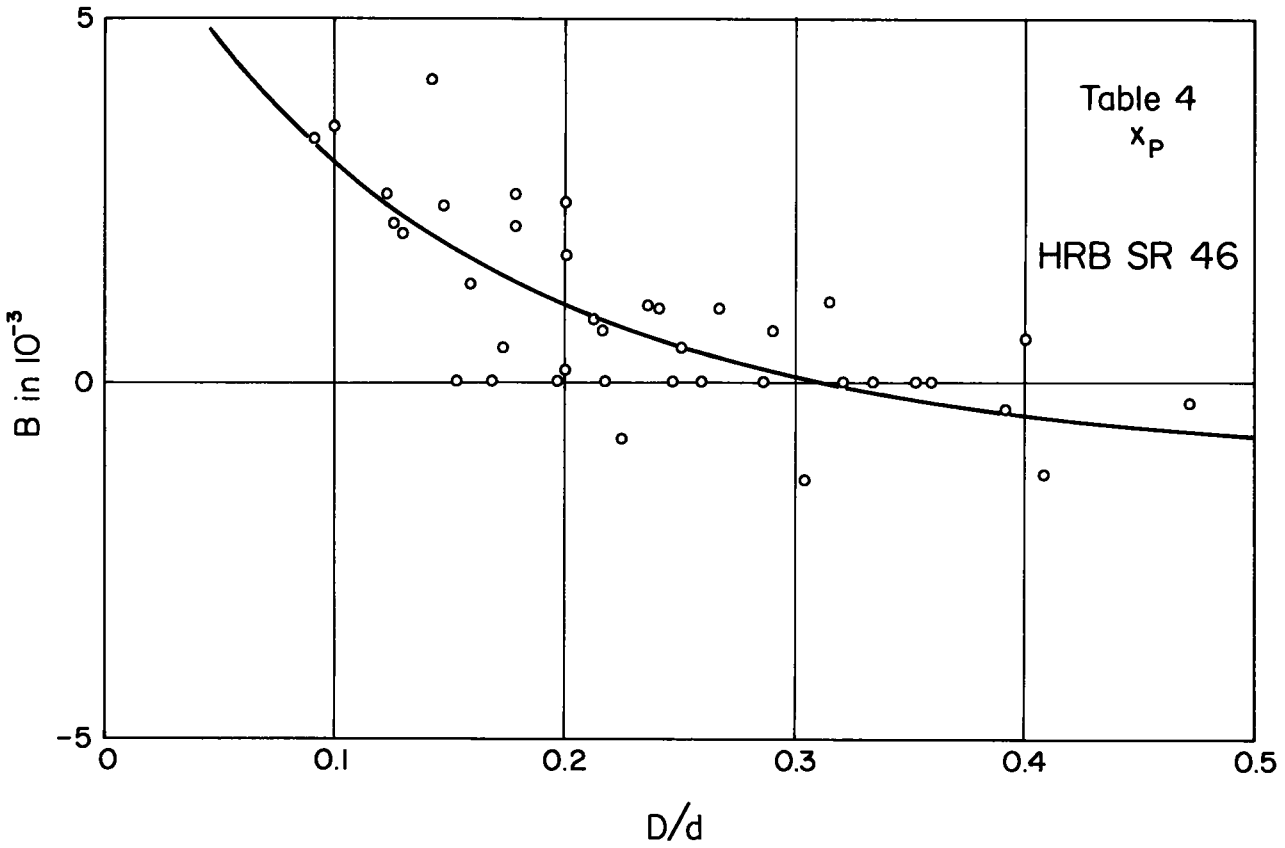

Figure B-17. Slope parameter vs thickness index parameter, pavement deflection.

$$\frac{d}{D} A = 2.5 \times 10^{-5.0 - 2.0 \log D/d} \qquad \text{(B-12)}$$

The straight line approximating the response of $\left[\frac{a\,b}{d^2} B + 0.005\right]$ versus D/d is shown in Figure B-21 and can be expressed as

$$\frac{a\,b}{d^2} B + 0.005 = 3 \times 10^{2.3\,D/d - 3} \qquad \text{(B-13)}$$

Eqs. B-12 and B-13 may be solved for A and B, respectively, and combined with a form of Eq. 32 to yield the load-deflection response for the subgrade deflection data as

$$\frac{p}{q_N} = \frac{\left[\frac{d + 2a + 2b}{d}\right]^2 \left[\frac{x_g}{d + 2a + 2b}\right]}{2.5\frac{D}{d} \times 10^{-5-(2\log D/d)} + \frac{d^2}{a\,b}\left[3 \times 10^{2.3D/d-3} - 0.005\right]\frac{x_g}{d + 2a + 2b}} \qquad (41)$$

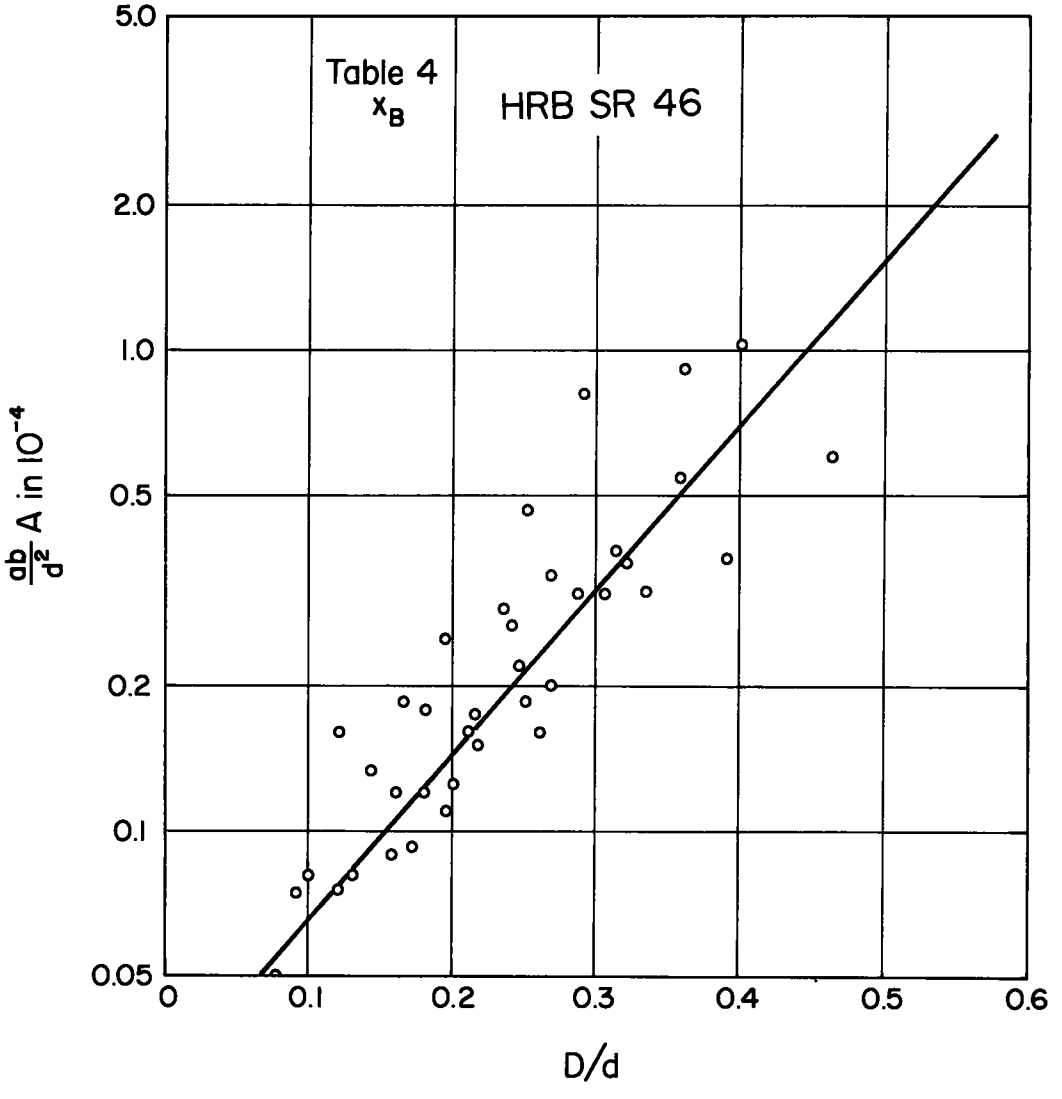

Figure B-18. Intercept parameter vs thickness index parameter, base course deflection.

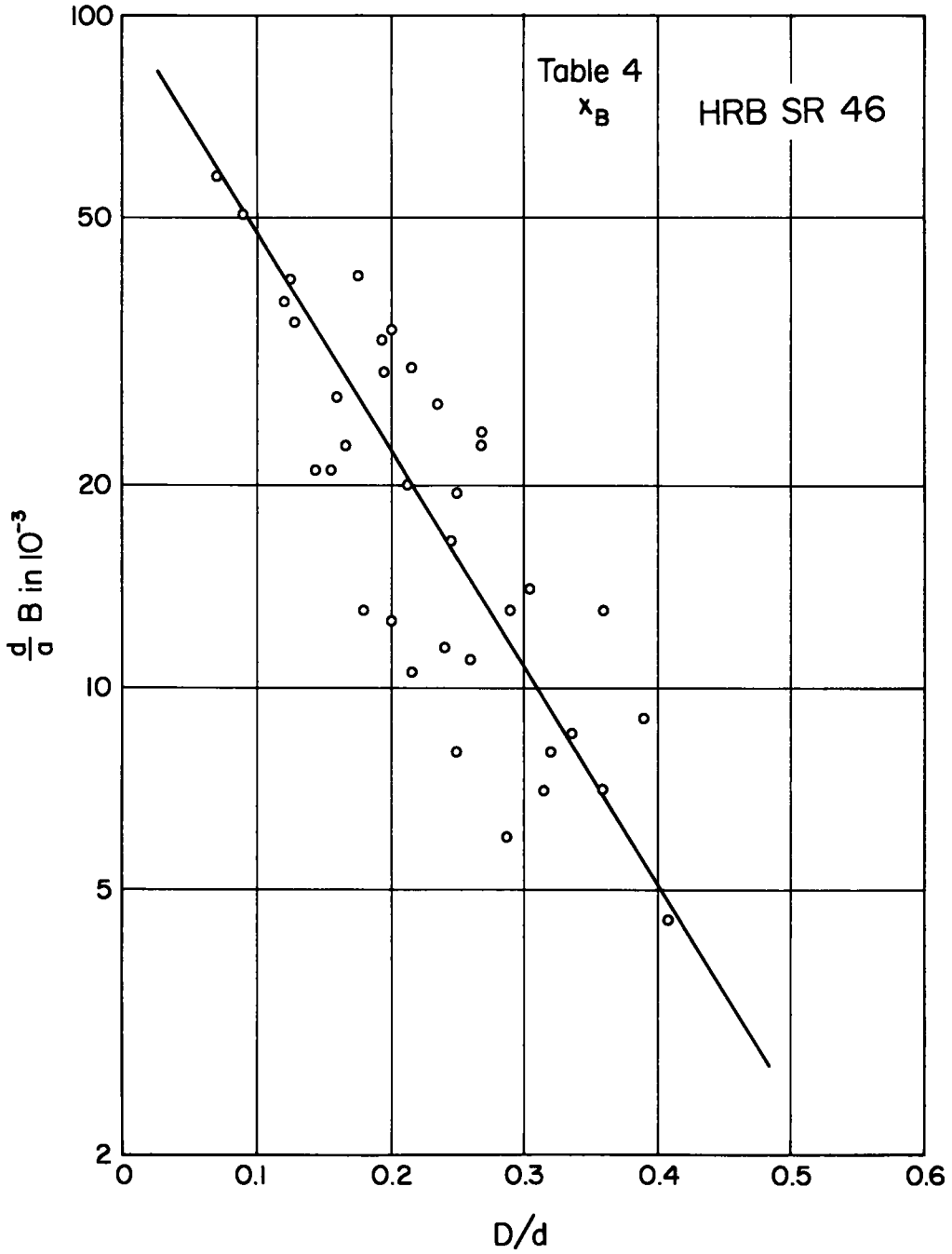

Figure B-19. Slope parameter vs thickness index parameter, base course deflection.

Temperature Effect

The pavement deflections in Table 6 of HRB SR 46 are analyzed in this section to determine the effect of temperature on response characteristics. Although fewer data are involved in this portion, the test procedure, pavement thicknesses, and plate diameters are generally similar. The only essential difference between these data and the data obtained from Table 4 of HRB SR 46 is temperature; Table 6 data were obtained at approximately 45 ± 5 F, whereas Table 4 data were measured around 84 ± 6 F.

The hyperbolic coefficients A and B are obtained as functions of D/d by means of Figures B-22 and B-23. The equation for the straight line approximating the A relationship shown in Figure B-22 may be written

$$\frac{ab}{d^2} A = 3 \times 10^{(2.7\, D/d)-6} \qquad \text{(B-14)}$$

while the equation for the curve approximating the B versus D/d data shown in Figure B-23 may be expressed

$$B = 12 \times 10^{(-2.5\, D/d)-3} \qquad \text{(B-15)}$$

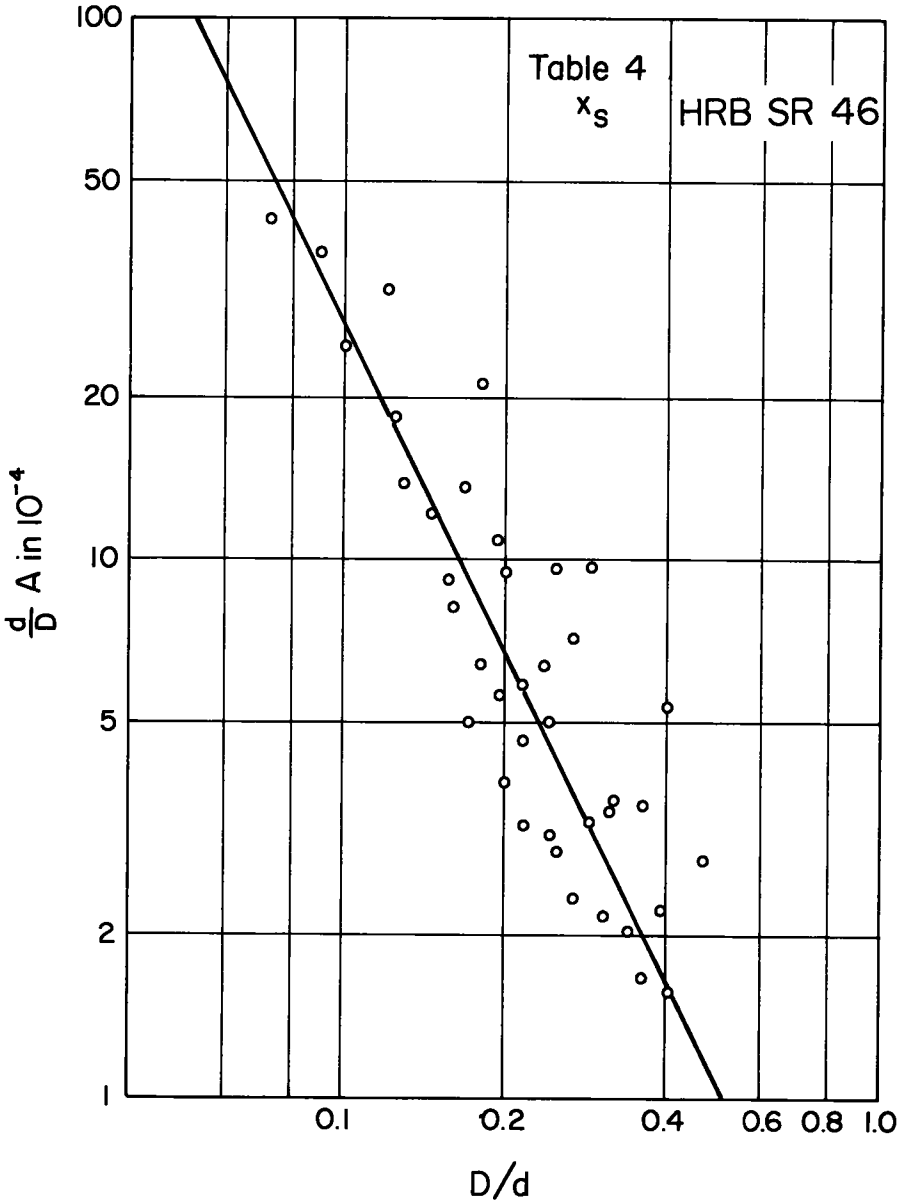

Figure B-20. Intercept parameter vs thickness index parameter, subgrade deflection.

Eqs. B-14 and B-15 may be combined with Eq. 32 to yield a general relationship for these data given by

$$\frac{p}{q_N} = \frac{x_p/d}{\frac{3d^2}{a\,b} \times 10^{(2.7\,D/d)-6} + \left[12 \times 10^{(2.5\,D/d)-3}\right] x_p/d} \quad (42)$$

The variations in the coefficients of Eqs. 39 and 42 probably may be attributed to the temperature difference between the two sets of tests. For example, the ratio between the A values calculated from Eqs. B-9 and B-14 reveals that the initial slope for the load-deflection data corresponding to the lower temperature (Table 6) will be approximately $1.33 \times 10^{0.63\,D/d}$ greater than that for the higher temperature data (Table 4). For $D/d = 0.3$ this ratio of initial slopes would be approximately 2.

Effect of Repetitional Loading

To a limited degree the effect of repetitional loading can be studied by examining the data recorded in Table 8a of HRB SR 46 (*4*). The most severe limitation is imposed by the fact that only 75 repetitions are conducted for any given situation. The Table 8a data were obtained by a cyclic load-and-release technique. A pressure of 16 psi was applied to the bearing plate; after the deflection measurement was taken, the load was released. In similar fashion pressures of 32, 48, and 64 psi were applied one cycle each. Following the release of the 64-psi pressure, an 80-psi load was applied and released 75 times; deflection measurements were recorded after 1, 10, 40, and 75 applications. In general, the data in Table 8a appear to be more erratic

than those in Table 4, and more difficulty was experienced in the analysis of Table 8a data.

First of all, the single-cycle data corresponding to applied pressures of 16, 32, 48, 64 and 80 psi are analyzed in a manner similar to the preceding approach. The relationships for the coefficients A and B are plotted in Figures B-24 and B-25; the respective equations approximating these data may be written as

$$\frac{a\,b}{d^2}A = 1.6 \times 10^{(5.8\,D/d)-6} \qquad \text{(B-16)}$$

and

$$B = 0.006 - 0.037\,D/d \qquad \text{(B-17)}$$

Combining Eqs. B-16 and B-17 with Eq. 32 gives a general equation describing these data which may be written

$$\frac{p}{q_N} = \frac{x_P/d}{\frac{1.6d^2}{ab} \times 10^{5.8\,D/d-6} + [0.006 - 0.037\,D/d]\,x_P/d} \qquad (43)$$

The differences between the coefficients in Eq. 43 and Eq. 39 are most probably attributable to the difference in test procedure in obtaining the two sets of data. Using Eqs. B-9 and B-16, the ratio of the initial slopes of the load-deflection plot for the Table 8a data to the Table 4 data may be found to be $2.5 \times 10^{-2.47\,D/d}$; for $D/d = 0.3$, this ratio becomes approximately one-half. A pronounced difference is noted between the values for B as obtained from Table 4 data and from Table 8a data. As shown in Figure B-17, Table 4 values for B tend to become slightly negative as D/d increases; however, Table 8a values for B, as shown in Figure B-25, reveal that a large number of tests manifested significantly negative slopes. A negative value for B implies that resistance to deformation is increasing more rapidly than predicted by a linear load-deflection relationship, or the pavement becomes stiffer with increasing load. As can be seen, test procedure does manifest a significant effect on the data obtained.

The approach employed in analyzing the effect of repetitional loading was to first determine a dimensionless factor,

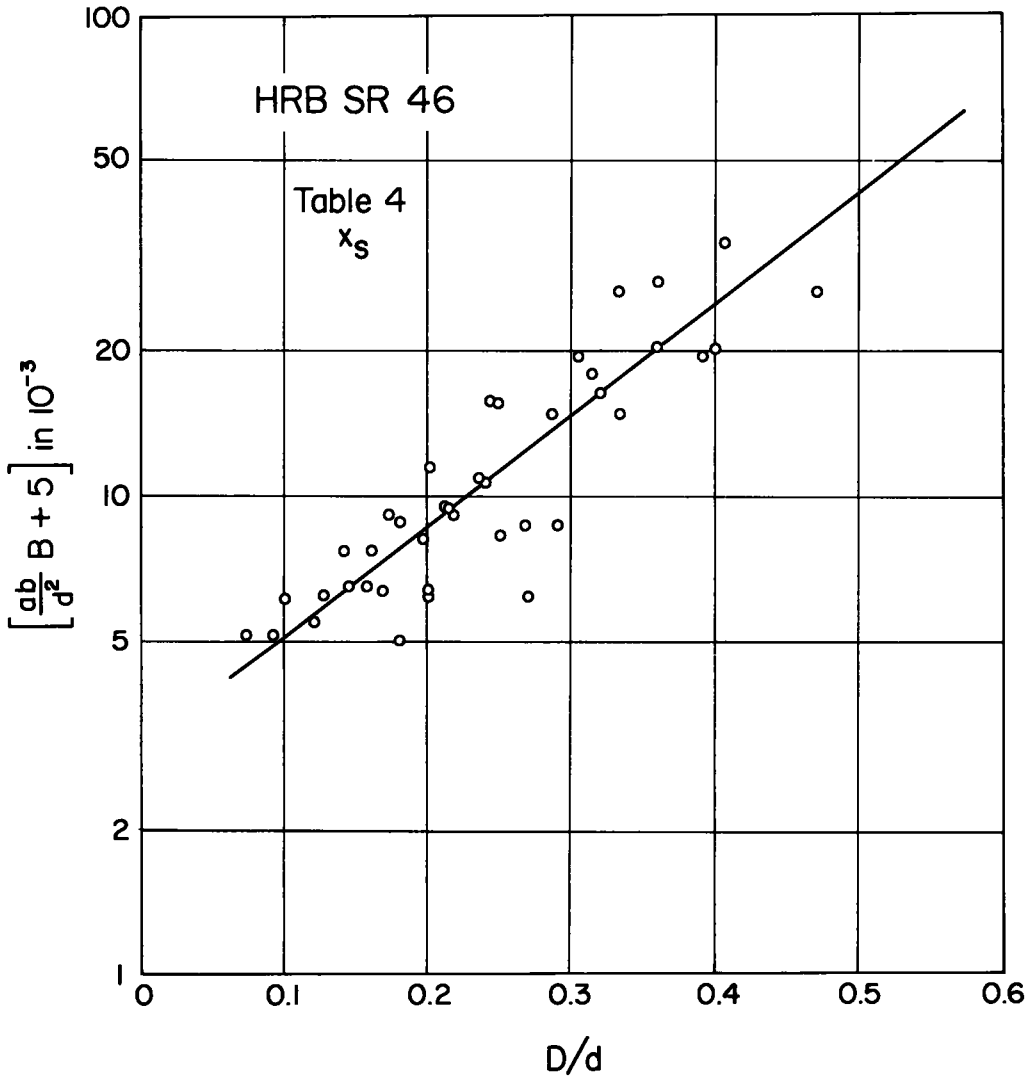

Figure B-21. Slope parameter vs thickness index parameter, subgrade deflection.

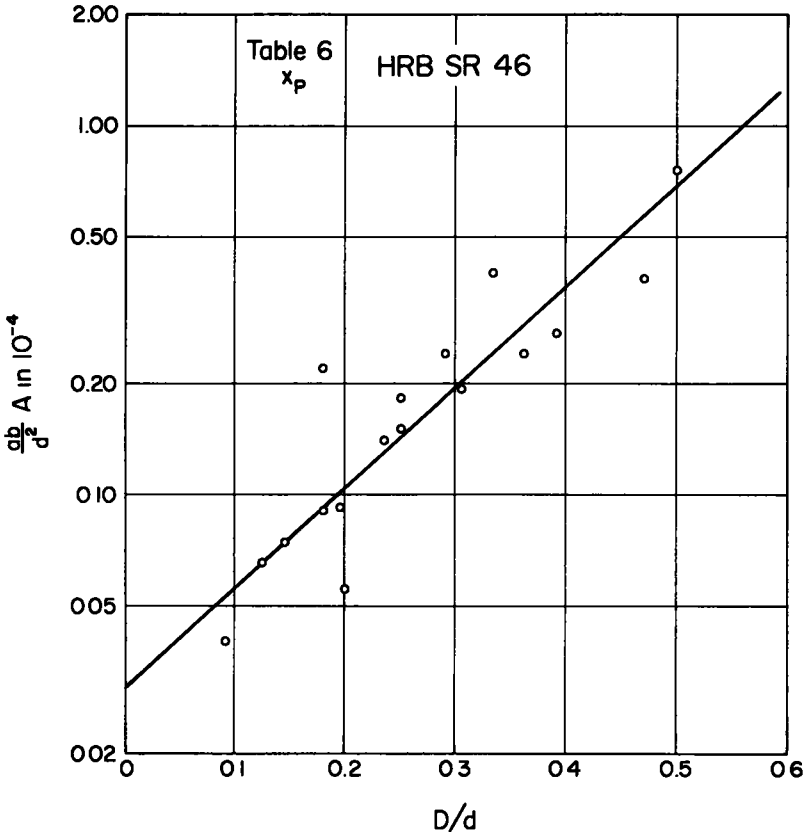

Figure B-22. Intercept parameter vs thickness index parameter, pavement deflection.

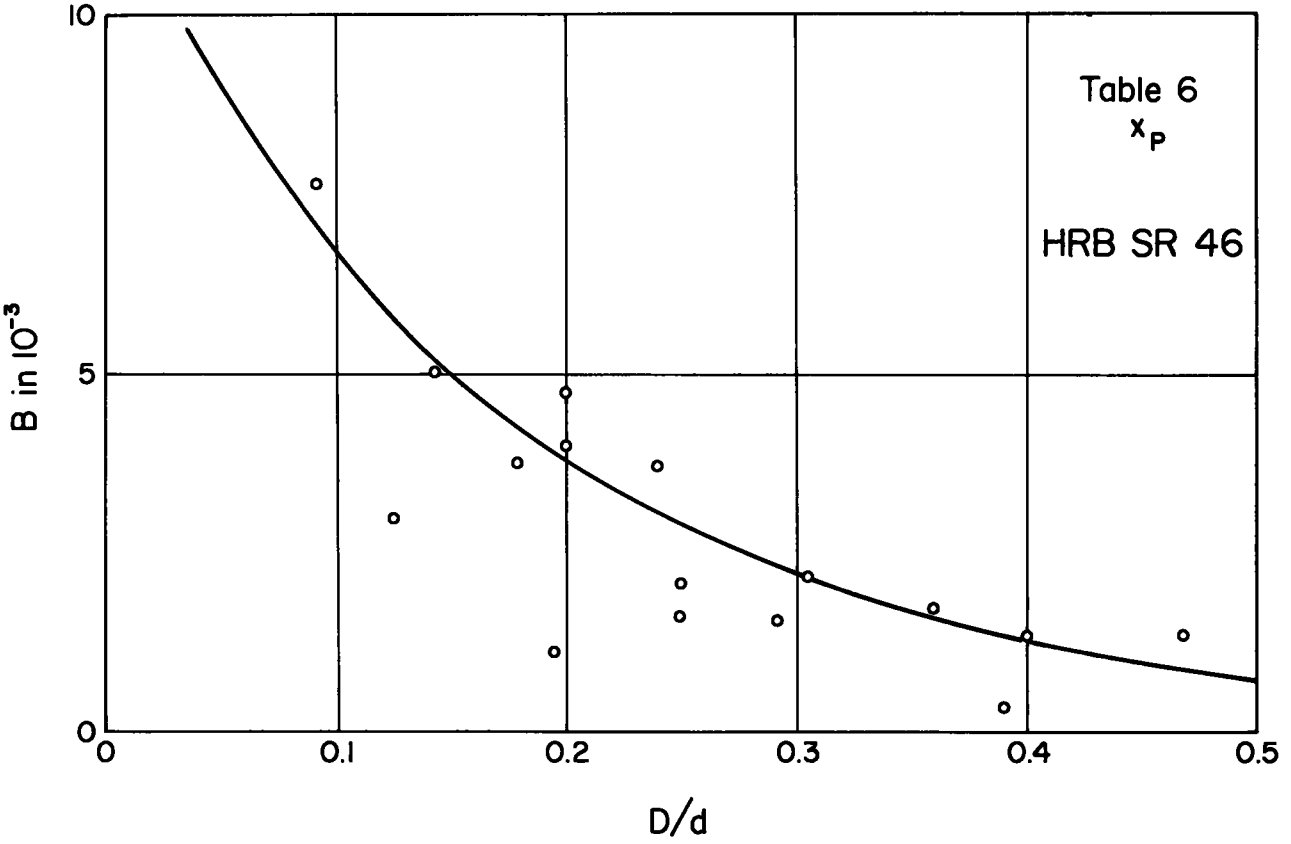

Figure B-23. Slope parameter vs thickness index parameter, pavement deflection.

f, ratioing the deflection at N applications to the deflection at one application for the 80-psi load. This factor was obtained for 10, 40, and 75 load applications. Transformed hyperbolic plots of the dimensionless parameters $\frac{N}{(f-1)}$ and N were made for all cases. In general, these data yielded quite good straight lines; a typical example is shown in Figure B-26. Writing the equation of the straight line as

$$N/(f-1) = \alpha + \beta N \qquad \text{(B-18)}$$

the coefficients α and β must now be examined to determine their functional dependence on D/d. The coefficient α is plotted versus D/d in Figure B-27, which gives an approximate result as

$$\alpha = 28 \qquad \text{(B-19)}$$

Figure B-28 shows $\frac{a}{d}\beta$ plotted versus D/d; these data may be described by the straight line

$$\frac{a}{d}\beta = 1.44\, D/d - 0.04 \qquad \text{(B-20)}$$

Solving Eq. B-18 for the factor, f, and combining with Eqs. B-19 and B-20, the effect of repetitional loading for the limited case studied may be expressed as

$$f = 1.0 + \frac{N}{28 + [1.44\, D/d - 0.04]\, d/a\, N} \qquad (44)$$

The factor f is that number by which the single-cycle deflection must be multiplied to obtain the deflection after N cycles of the same load. For these data the single-cycle deflection may be obtained by solving Eq. 43 for x_P/d as the dependent term, as indicated by Eq. B-6; then $f\, x_P/d$ represents the deflection parameter after N cycles of the given load.

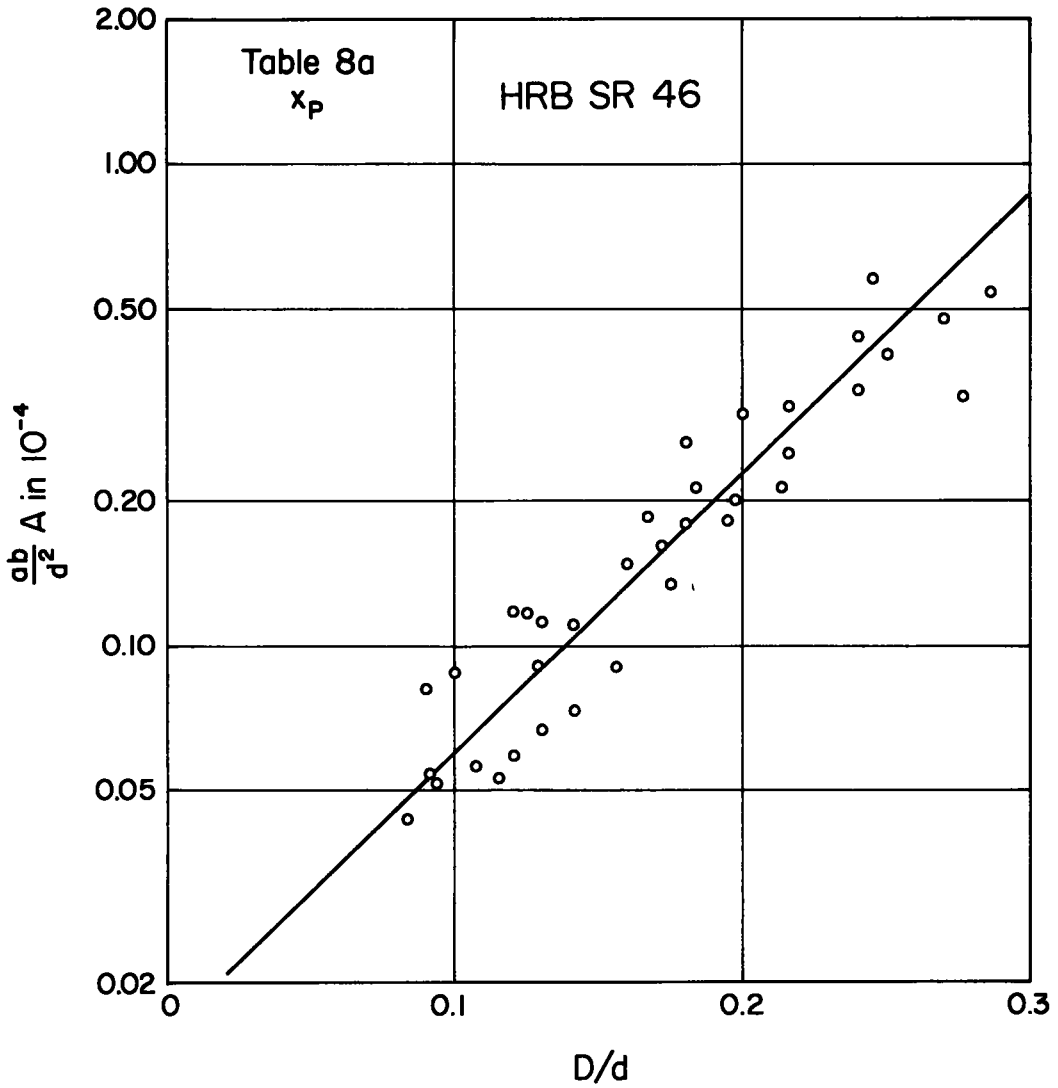

Figure B-24. Intercept parameter vs thickness index parameter, pavement deflection.

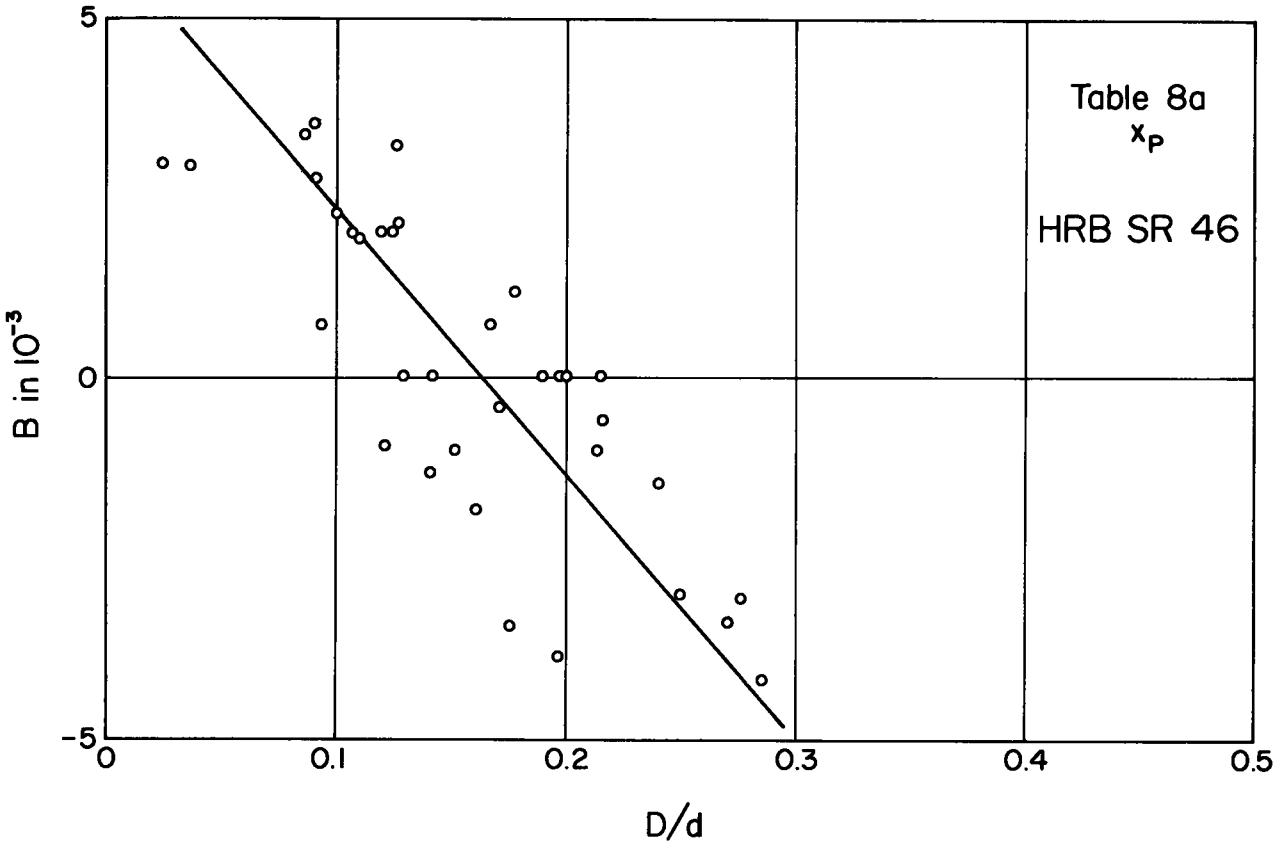

Figure B-25. Slope parameter vs thickness index parameter, pavement deflection.

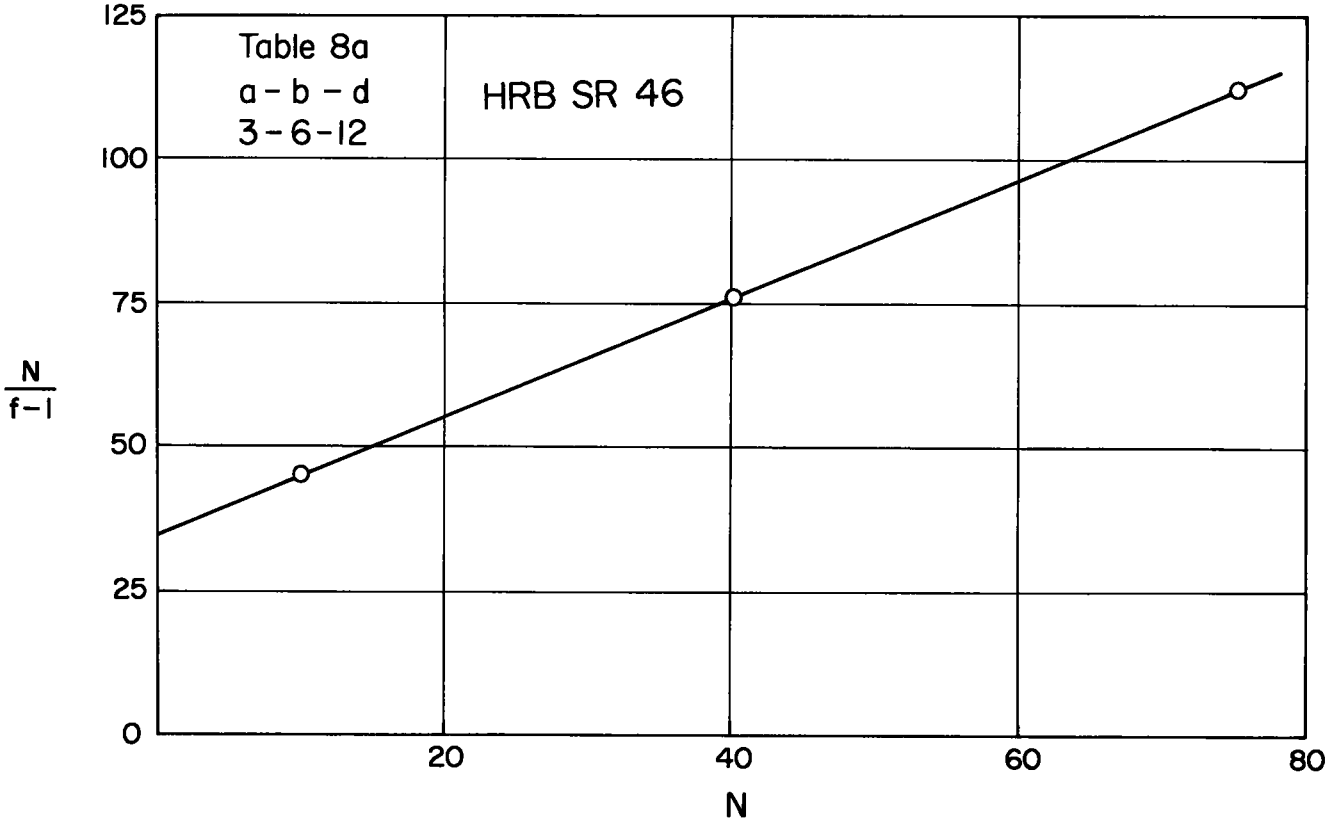

Figure B-26. Typical plot of repetitional deflection factor vs number of load applications, pavement deflection.

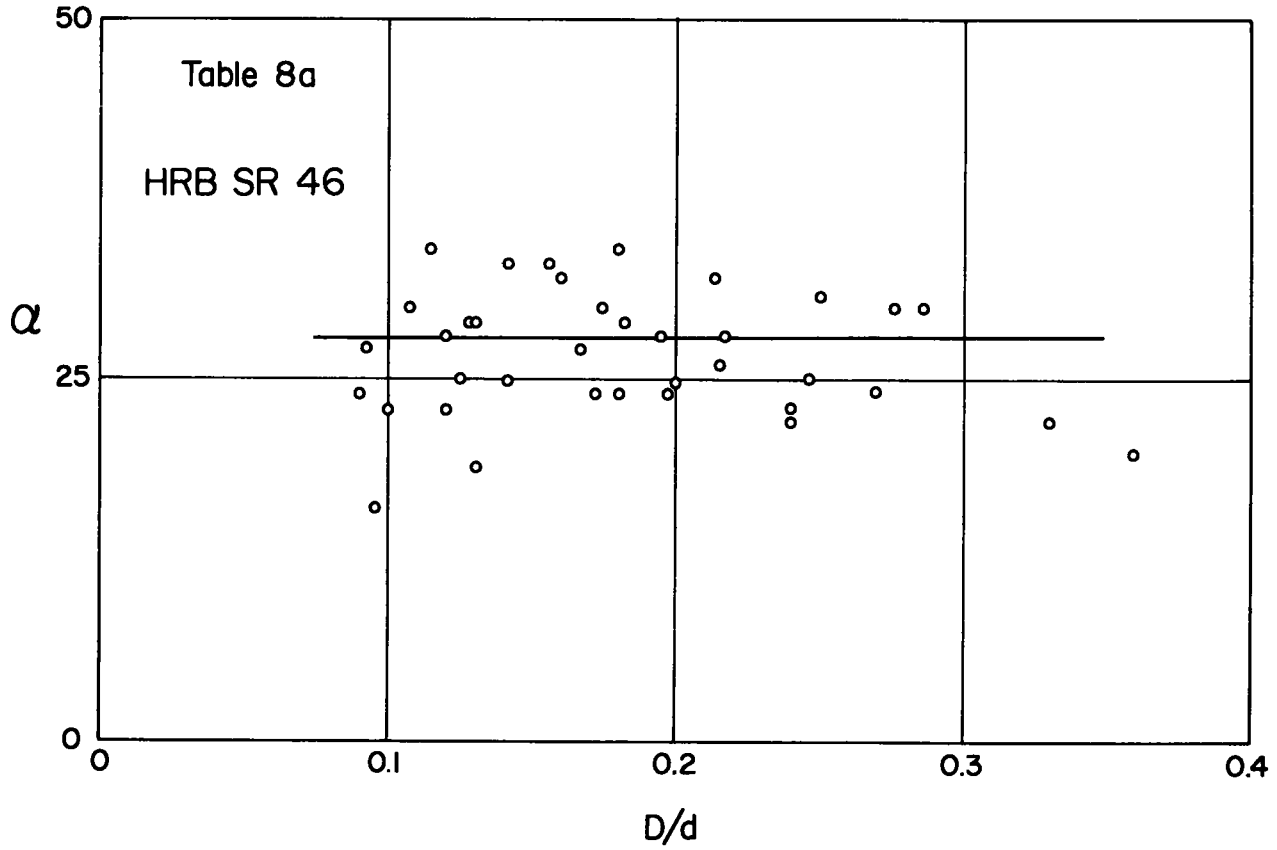

Figure B-27. Intercept parameter for repetitional loading vs thickness index parameter, pavement deflection.

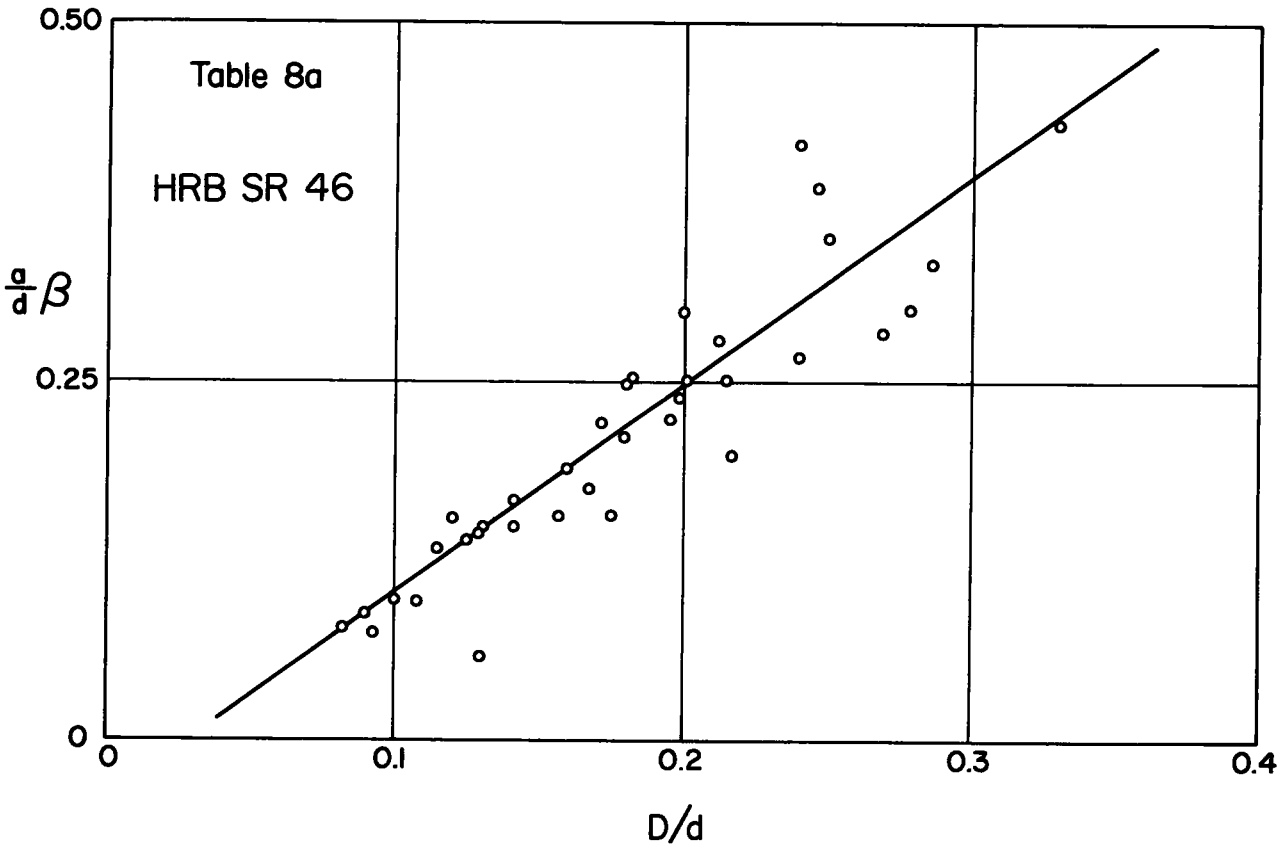

Figure B-28. Slope parameter for repetitional loading vs thickness index parameter, pavement deflection.

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