328

# HIGHWAY RESEARCH BOARD

## Bulletin 328

# *Pavement Roughness*

## Measuring Technique and Changes



National Academy of Sciences—
National Research Council

publication 100

# HIGHWAY RESEARCH BOARD

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## 1962

NRC HIGHWAY RESEARCH BOARD

**Bulletin 328**

# *Pavement Roughness*

## Measuring Technique and Changes

Presented at the
41st ANNUAL MEETING
January 8-12, 1962

National Academy of Sciences—
National Research Council
Washington, D.C.
1962

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# Contents

# Cumulative Changes in Rigid Pavements With Age in Service

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This paper presents the analysis of some 6,000 mi of pavement profile obtained by the Michigan Pavement Performance Study in the 3-year period of 1958-59-60. This mileage on the Michigan State trunklines consists largely of reinforced concrete pavement with service periods of up to 35 years, under a considerable range of traffic intensity, soil conditions, and climatic environment. Emphasis is given to pavement changes over long periods of time, but data are also available on a number of special features bracketing Michigan design conditions.

Procedures developed for evaluating pavement performance are based on two basic criteria for measuring pavement change. The first, or "roughness index," is riding quality based on pavement profiles and recorded in inches of vertical displacement per mile. The second is structural continuity, based on the cracking pattern, expressed as a continuity ratio, and defined as the ratio of uncracked slab length to an assumed slab length of 15 ft. Both of these measures are recorded on pavement condition surveys.

The data afford significant comparisons between current design for year-round service without seasonal load limitations, and older roads considered inadequate for present day traffic; the pavement study includes plain concrete, reinforced concrete of conventional design and joint spacing, and an experimental project with continuous reinforcement. The dominant effects of soil conditions, frost action, drainage, and other environmental conditions are revealed on certain projects where wider variation in these conditions has occurred. Construction control is revealed as an important factor in riding quality, with "built-in roughness" varying through wide limits. Subgrade preparation is also found to be an important influence on initial riding quality and subsequent performance. Differentials in performance between the traffic lane and the passing lane occur on certain projects, whereas on others there is no apparent effect of load application.

• FROM THE VERY BEGINNING pavements have been built simply to provide durable surfaces with improved riding qualities for the safety, comfort, and convenience of the highway user. With the advent of automotive vehicles with constantly increasing speed of travel, smoothness of the pavement surface or riding quality has become increasingly important in pavement design and construction.

Also from the very beginning, pavement builders must have gaged the success of their endeavors by experience and direct observation of their pavements under the service conditions to which they were subjected. Thus, pavement condition surveys, though they may not have been formalized as they are today by systematic procedures, are as old as the oldest pavements. In the development of modern highway systems, the importance of permanence of riding quality or durability has focused increased attention on the strength of the pavement structure and its ability to maintain structural continuity under the increased loads and mounting volume of modern traffic, in combination with the stresses and strains associated with environmental conditions.

The increased use of rigid concrete pavements to provide high quality surfaces has paralleled the rapid development of automotive transportation. The physical characteristics of such rigid pavements have led highway engineers naturally and logically to judge their performance by the rate at which they become rough and lose their riding quality and the rate at which they crack and lose their structural continuity. The point of these introductory remarks is simply to emphasize that the changing condition of rigid pavements, as reflected in cracking and roughness, has always been a natural and realistic measure of pavement performance. The major contribution of recent years has been the introduction of refinements in procedures for making pavement condition surveys and development of more precise criteria for evaluating pavement performance.

This approach must have been apparent to the Highway Research Board Committee on Rigid Pavement Design when it formulated, in January 1959, a research project entitled "Investigation of Existing Pavements." At that time it listed a number of significant changes in rigid pavement design in recent years and made the following statement:

> It is believed to be highly important to determine the effects of these changes in order to avoid the possibility of constructing miles of pavement which might otherwise fail prematurely. It is also believed that, in many respects, the pavements which are presently in existence constitute the only dependable sources of information on which to base future designs.

Well-organized pavement condition surveys in Michigan date back to the middle 1920's, when the late V.R. Burton organized a group of research workers who started a series of statewide pavement condition surveys including comprehensive data on soil conditions and climatic environment (1, 2, 3). This work has been carried on over the years by a number of individuals well-known among highway engineers and soil scientists, including Kellogg, Benkelman, Stokstad, and Olmstead.

These investigators early found significant correlation between pavement performance and environment, including soil type, drainage, and climatic factors, a viewpoint that has continued to exert a dominant influence on pavement design in Michigan. Improvements in this approach to pavement design have led to more accurate evaluation of soil conditions, drainage, and climatic environment; the utilization of local soil materials of favorable characteristics; and the selection of pavement structures that more fully utilize available subgrade support. Though many of these factors are uncontrolled variables, difficult to measure and perhaps impossible to express in a mathematical formula, current pavement performance studies in Michigan have been predicated on the belief that the integrated results of these uncontrolled variables could be measured quantitatively by more accurate field surveys and objective analysis of the results. Furthermore, it was felt that pavement performance, in terms of changes in the pavement profile and cracking pattern, could be expressed numerically by a roughness index and a continuity ratio.

The first attempt in Michigan to measure pavement performance quantitatively in terms of pavement roughness and structural continuity was a cooperative investigation, initiated in 1952, involving the Michigan State Highway Department, the University of Michigan, and the Wire Reinforcement Institute. The data obtained from that investigation, carried on over a period of five years, indicated that pavements with steel reinforcement were measurably smoother and that cracking was measurably less than in the unreinforced concrete pavements involved in that investigation. Of perhaps more importance to the present discussion was the fact that pavement performance was evaluated in terms of a continuity ratio related to the cracking pattern and a roughness index based on measured vertical displacement in the pavement profile.

## MICHIGAN PAVEMENT PERFORMANCE STUDY

In further pursuit of these objectives, a cooperative investigation was next undertaken in 1957 by the University of Michigan, the Michigan State Highway Department, and sponsors representing the trucking industry (the Michigan Trucking Association, the American Trucking Associations, Inc., and the Automobile Manufacturers Association). Pavement performance studies under this sponsorship continued for approximately two years, and in July 1959, were taken over by the Michigan State Highway Department as part of the Michigan Highway Planning Survey—Work Program financed by HPS funds, under the supervision of the Bureau of Public Roads. This program has continued to date under a contract with the University of Michigan. In the first two years, equipment for recording pavement profiles was developed and tentative procedures were established for evaluating the data obtained.

A truck-mounted profilometer for accurately recording pavement profiles was built, closely following similar equipment used for some time by the California State Highway Department. Electronic recording equipment and integrators were added to provide a chart-recorded profile of the pavement in both wheelpaths and to compile the cumulative vertical displacement in inches per mile. Means were also provided to record pavement cracks and joints. Early results from these studies (4) were presented to the Highway Research Board in January 1959. There is nothing new about profilometers and measuring roughness as the sum of vertical displacements per mile as a roughness index. In his paper to the Highway Research Board in January 1960, Hveem (5) presented an interesting review of this subject and described a number of such devices, the earliest one in available records dating back to before 1900.

### Pavement Profiles for 1958-1960

During the three years 1958, 1959, and 1960, close to 6,000 lane-mi of pavement profile were accumulated. The analysis of these data has proceeded concurrently, insofar as personnel and facilities would permit; it is the purpose of this paper to present some of the significant results presently available, particularly to cumulative change in rigid pavement profiles with age in service. It is felt that the data reveal significant trends in pavement performance and direct relationship to controlling design conditions. There is a tremendous volume of information involved in some 6,000 mi of pavement profile; therefore, the present discussion is limited to several classifications of rigid pavement that have been sampled in sufficient quantity to provide a reasonable basis for analysis. All pavements included in the profile survey are part of the Michigan State trunkline system of some 9,435 mi, including 8,050 mi of two-lane pavement, 135 mi of three-lane pavement, and 1,250 mi of divided four-lane pavement. The trunkline system thus amounts to 21,500 lane-mi of pavement; thus, the three years of profile surveys discussed in this report provide a sample of approximately 27.5 percent of the total trunkline mileage.

Most of the data obtained are for pavements rated as Class 1 and Class 2, although some data are presented from surveys of Class 3 and Class 4 pavements. In this connection, it is necessary to define these four pavement classes as they have been incorporated in the Michigan trunkline surveys. The first pavement evaluation of the Michigan trunkline system was presented as of January 1, 1958, and was compiled from the Michigan State Highway Department's records, including design data, pavement condition surveys, maintenance records, and soil surveys. At that time, 55 percent of the State trunkline system was rated as Class 1 and Class 2 roads, adequate for legal axle loads at all seasons of the year; 45 percent was rated as Class 3 and Class 4, inadequate and requiring Spring load restrictions. In this road classification for legal axle loads, the four levels of adequacy selected are defined as follows:

Class 1.—No seasonal restrictions. Pavement and subgrade adequate for year-round service, as represented by natural sand and gravel subgrades with superior natural drainage.

Class 2.—No seasonal restrictions. Pavement designs that compensate for seasonal loss of strength, as represented by the subgrades of fine-grained soils and generally inferior drainage corrected by the use of free-draining sand and gravel subbases, raising grade line to improve drainage, removal of frost-heave soils.

Class 3.—Spring load restrictions required. Pavement designs that do not compensate for seasonal loss of strength, as represented by fine-grained soils, susceptible to frost-heaving and pumping, and with inadequate drainage provisions.

Class 4.—Spring load restrictions required. Pavement designs inadequate for legal axle loads at all times, as represented by older roads completely deficient and requiring continuous maintenance to provide year-round service for legal axle loads.

The data selected from 6,000 mi of pavement profile, for the present discussion, are shown on a series of charts developed as a standard format after considerable "cut-and-try" experimentation. In these charts, as shown in Figure 1, the roughness index in inches of vertical displacement per mile is plotted as the horizontal abscissa and the years in service as the vertical ordinate. The data in Figure 1 are for the traffic lane of Class 1 rigid pavements and represent pavement profiles of some 556 mi of pavement. At the top of the chart is a tentative roughness rating that has been in use for several years (4). Each plotted point represents an individual pavement profile in the outer or inner wheelpath of a specific construction contract. There are 123 such contracts and 556 mi of pavement included in the figure—when only one lane of a contract has been surveyed, there will be two plotted points for that contract; when both traffic lanes have been surveyed, there will be four such points for that contract. In general, it has been found that the outer wheelpath is rougher than the inner wheelpath; and though special studies have been made of this variation, these studies are discussed in detail in this paper.

## Significant Variations in Pavement Profiles

In spite of the wide scattering of roughness index values in Figure 1, there are a number of characteristics of these data, the analysis of which indicates significant trends in pavement performance:

1. There is a general increase in roughness with age which will be discussed as evidence of cumulative changes in rigid pavements with age in service.
2. There are a number of specific projects that exhibit roughness indices much less than the general trend and others with values much greater, both of which may be related to controlling design or construction conditions responsible for this abnormal behavior.
3. There is a discontinuity in average roughness indices at a service period of approximately 25 years which may be related to construction changes in the trunkline system and indirectly influenced by Michigan design procedure.

## Cumulative Changes in Pavement Profiles

Evaluation of cumulative changes in rigid pavements with age in service is the major objective of this paper. The method of evaluation proposed after considerable study is to establish a band of normal behavior, as shown in Figure 1. The first step is to compute the average roughness index for each 5-year period as the center of gravity of all observations in that period. These averages are shown as a triangle on the chart, through which the central line of the normal behavior band is drawn. For the first 25 years, the average results fall consistently along a line with an intercept of 65 in. per mi on the horizontal axis and a slope of 4.5 in. per yr as the average increase in the roughness index. After 25 years there is a discontinuity or displacement in this average line, which is discussed later.

The width of this band of normal behavior has also been established by trial and error as parallel lines that bracket 85 percent or more of the observations and balance the excluded observations, indicating abnormal performance, on either side of the band. For example, in Figure 1, for observations in a period of less than 25 years, approximately 7.5 percent of the pavement profiles have roughness indices less than normal and the same approximate percentage, greater than normal. Projects falling outside this band represent abnormal behavior, which may provide the most informative data available

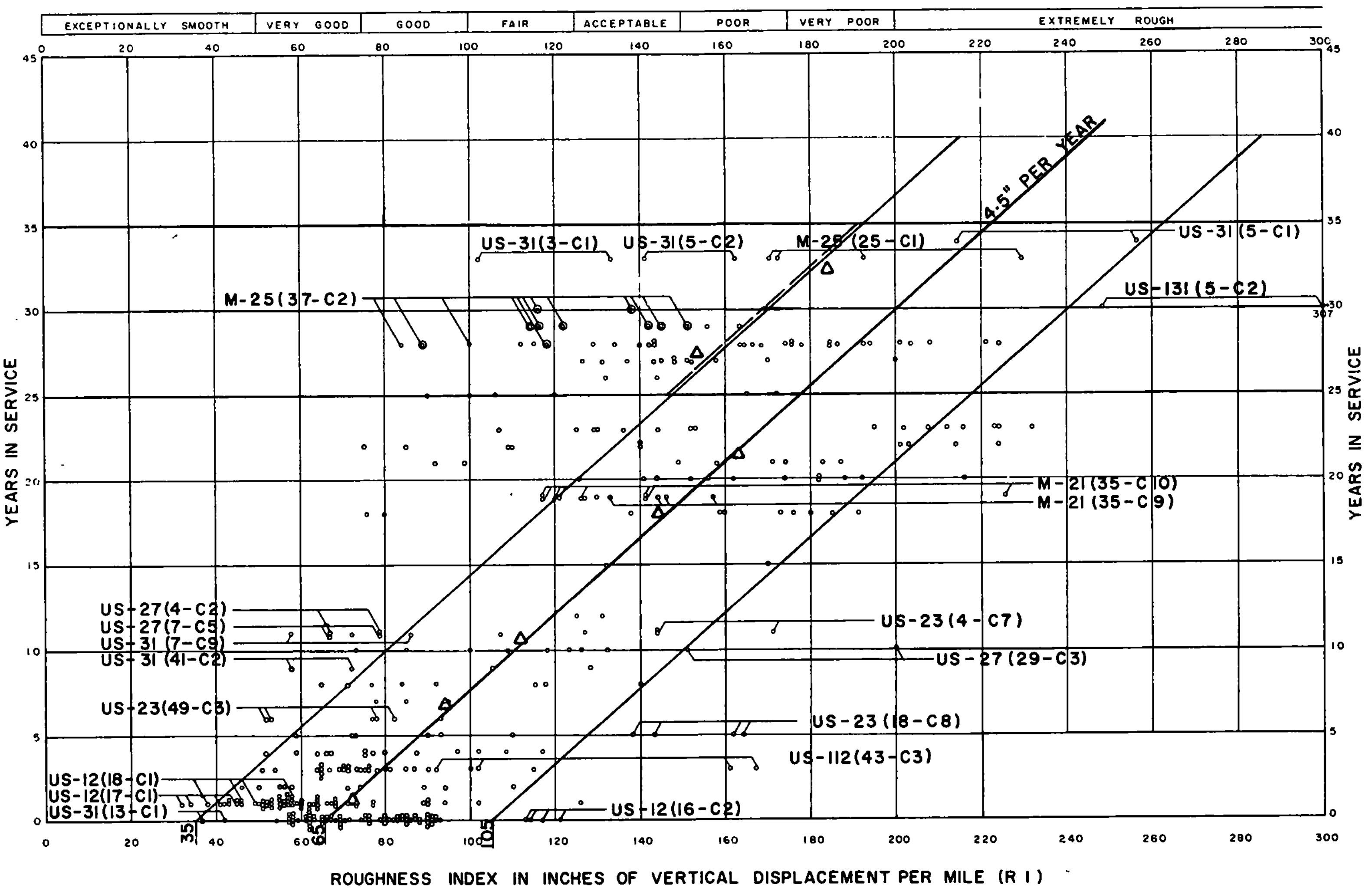


Figure 1. Roughness vs years in service, Class 1, traffic lane, based on 1958-59-60 surveys.

to relate pavement performance to design and construction conditions. Examples are cited later in a discussion of abnormal behavior.

The important deduction from these data at this point is that rigid pavements represented by this sample suffer a continuing or cumulative increase in roughness at an average rate of about 4 to 5 in. per mi per yr. The fact that these pavements have been rated as Class 1 pavements implies that they are adequate or more than adequate for legal axle loads at all seasons of the year; thus, load-carrying capacity is not a controlling factor in this progressive loss of riding quality. From this it follows that environmental factors, mainly associated with seasonal cycles, dominate this type of pavement deterioration, and there is other evidence to support this conclusion.

## Older Pavements Showing Abnormal Behavior

There are a number of projects in Figure 1 with roughness indices substantially less or greater than those within the band selected to represent normal behavior. In the first place, there is the definite discontinuity in the 5-year averages, influenced by a grouping of a number of projects with roughness indices much less than those within the band of normal behavior deduced from pavements less than 25 years old.

The first and perhaps major factor in this shift arises from the fact that many of the older pavements built before 1936 have been retired from service by reconstruction, recapping, or a change in classification. With a few exceptions, only those projects exhibiting superior performance are still in service and have been picked up in the profile surveys. A review is in progress to trace the history of all concrete pavements built before 1936, but complete results are not available for this report. Consequently, only a few examples can be cited at this time to illustrate this point and to indicate the close correlation between design and construction conditions and unusual pavement performance.

One such project, US-31 (3-C1), after 33 years of service, shows exceptionally good performance, with a roughness index falling well below the band of normal behavior and a riding quality still rated fair to acceptable. Another project, US-31 (5-C1), after 34 years of service, is rated extremely rough, with a roughness index falling near the upper limit of the band of normal behavior. Both were rated as Class 1 pavements on the basis of an area soil survey identifying the soil type as Plainfield sand, a superior subgrade with high internal stability and excellent drainage. These two projects, within several miles of each other, were built by the same contractor and have closely comparable traffic. The soil classification of Plainfield sand is correct for the project showing superior performance, but incorrect for the second project, which has become extremely rough. In the latter case, the project is located at a transition in soil types, the major portion being on a silty clay loam with inferior drainage conditions; this part of the pavement should have been rated as Class 3 or Class 4. The transition in soil types and marked changes in pavement performance are accurately identified on the pavement profile.

Another revealing example that may be cited is a 33-year-old project, US-31 (5-C2), which appears to have shown much better than average performance. A review of the pavement profile shows that this contract covers an area of well-drained sands of the Plainfield or closely related series, but with several smaller areas of low-lying poorly drained soils. These areas became extremely rough after some 30 years of service, but were recapped with a bituminous surface in 1956. The balance of the pavement had become quite rough, with considerable cracking shown by a reduction in the continuity ratio from 6.66 to 1.35. The roughness indices for this pavement, which is still in service, were reported in Figure 1 as the average for the entire project. Segregation and reclassification of the sections that became very rough and have been resurfaced and correction of the roughness index for the balance of the pavement would bring it more nearly within the band of normal behavior, with slightly better than average performance.

Another group of points showing better than normal performance are identified as Contract M-25 (37-C2). This pavement is on a shoreline road along Lake Huron built along a beach ridge on soil identified as the Eastport series, another high-quality subgrade. This is the clue to its superior performance, but it has nevertheless increased

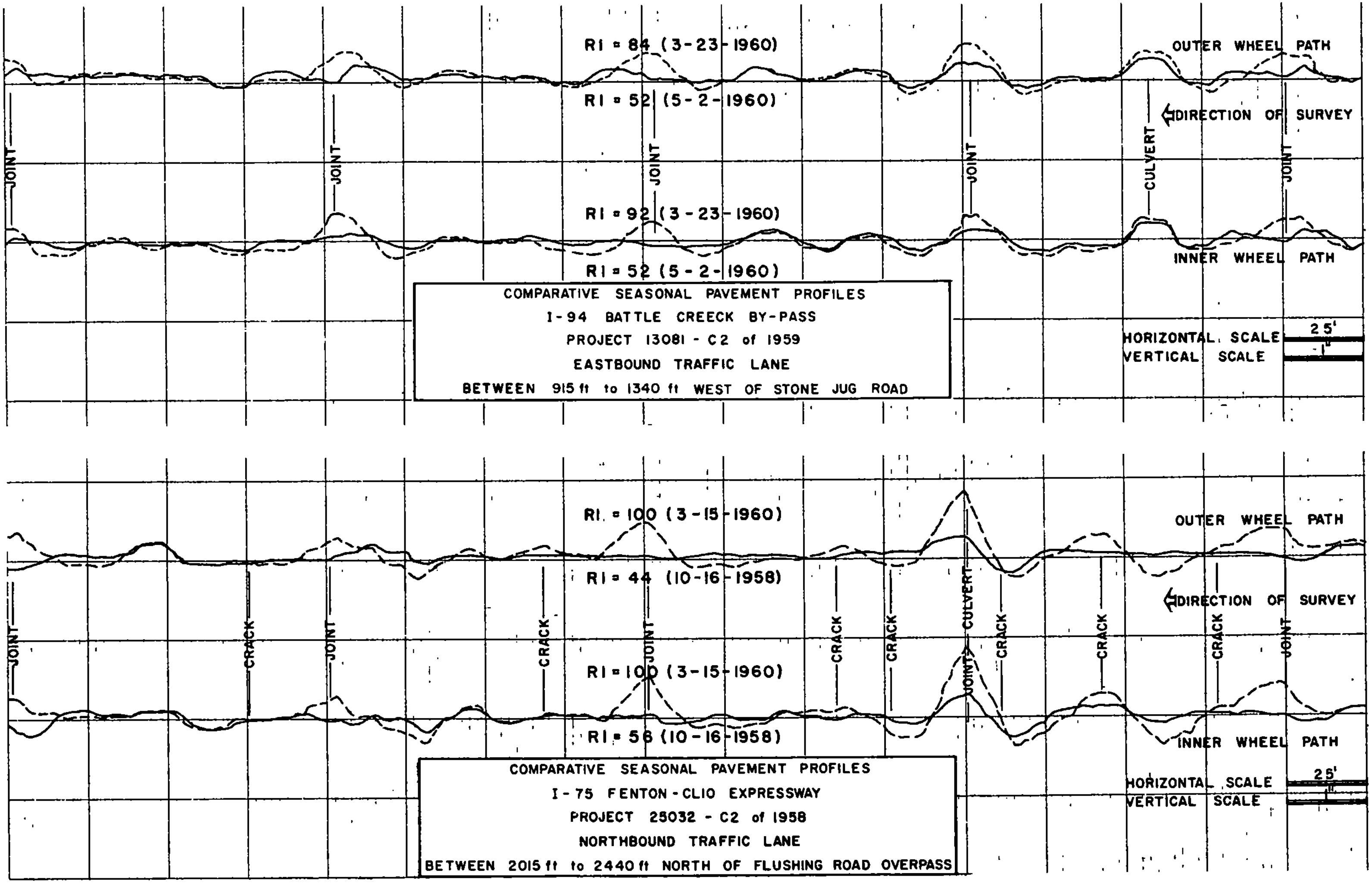


Figure 2. Comparative seasonal pavement profiles.

in roughness, although at a reduced rate, as residual displacement results from seasonal frost action, details of which will be discussed later.

Another example, shown in Figure 1, is Contract US-131 (5-C2), the only project in the survey more than 25 years old that is still in service, even though its roughness index is much greater than those within the band of normal behavior. Roughness indices of 248 in the inner wheelpath and 307 in the outer wheelpath show that it has become extremely rough and is actually beyond the limits of the tentative rating scale. This project is a plain concrete pavement built in 1929 just south of the city limits of Cadillac. In addition to being rough, it is badly cracked with its continuity ratio having been reduced from 6.67 to 0.85. In terms of average slab length, this is a change from 100 ft between joints to an average of about 12 ft between cracks.

As a whole, this old project is the product of outmoded design standards that no longer represent Michigan practice. The rougher sections are over inferior-type soils that would now be compensated for by raises in grade and a free-draining granular subbase. A substantial part of this project was in a relocation over generally good subgrade soils, but was built without adequate control of subgrade compaction and without the advantage of stage construction to condition the subgrade before paving. This project is on a major route and would doubtless have been rebuilt some time ago except that its relocation, long-planned, was postponed until the advent of the major improvement program of the past several years. This relocation is now in progress and only this unusual set of circumstances found it still in service when the profile survey of this route was made in 1959.

## Younger Pavements Showing Abnormal Behavior

Pavements less than 25 years old shown in Figure 1 also include about an equal number of projects with roughness indices substantially above and below the limits of normal behavior. In the earlier years, the conclusion is quite inescapable that the wide range in riding quality must have been produced during construction and is thus initial or built-in roughness.

Looking first at the group of eight projects less than 15 years old with superior riding quality, there are three projects, US-31 (13-C1), US-12 (17-C1), and US-12 (18-C1), surveyed the year they were built or one year later, with roughness indices of 50 or less, which would be rated as exceptionally smooth. There are five additional projects, US-23 (49-C3), US-27 (4-C2), US-27 (7-C5), US-31 (41-C2), and US-31 (7-C9), which, allowing for normal increase in roughness, must have been built with an initial roughness in the same range and rated as exceptionally smooth. Five of these eight projects were built by two contractors who have gained special recognition for high quality workmanship. The same may be said for the other projects giving evidence of good workmanship, even though the illustration lacks the emphasis of repetitive coincidence of contractor and excellent performance.

Attention is next directed to a group of five projects less than 15 years old with roughness index values greater than those within the band of normal behavior. Again it may be assumed that normal increase in roughness would leave built-in roughness as the major source of decreased riding quality. Project US-12 (16-C2) was built in 1960 and surveyed in December of the same year. The low temperature may have produced some curling; but, other than this, there are no known job conditions contributing to the increased roughness other than contractor performance.

Project US-112 (43-C3) presents a particularly interesting comparison in that its westbound lane has a roughness index of 167 in the outer wheelpath and 161 in the inner wheelpath, rated as "poor." The eastbound lane, on the other hand, had a roughness index of 102 in the outer wheelpath and 92 in the inner wheelpath, which, while not outstanding, would at least be rated as "good." A report from the project engineer on this contract reveals that there were special job conditions which account for the abnormal result. The westbound lane was paved late in the year to provide for traffic during the winter, until the project was completed, and a considerable portion of it was hand finished. The eastbound lane was completed the next spring and was machine finished. Incidentally, the paving was done by the contractor who paved three of the eight projects previously cited as evidence of high quality workmanship. The pavement was a short

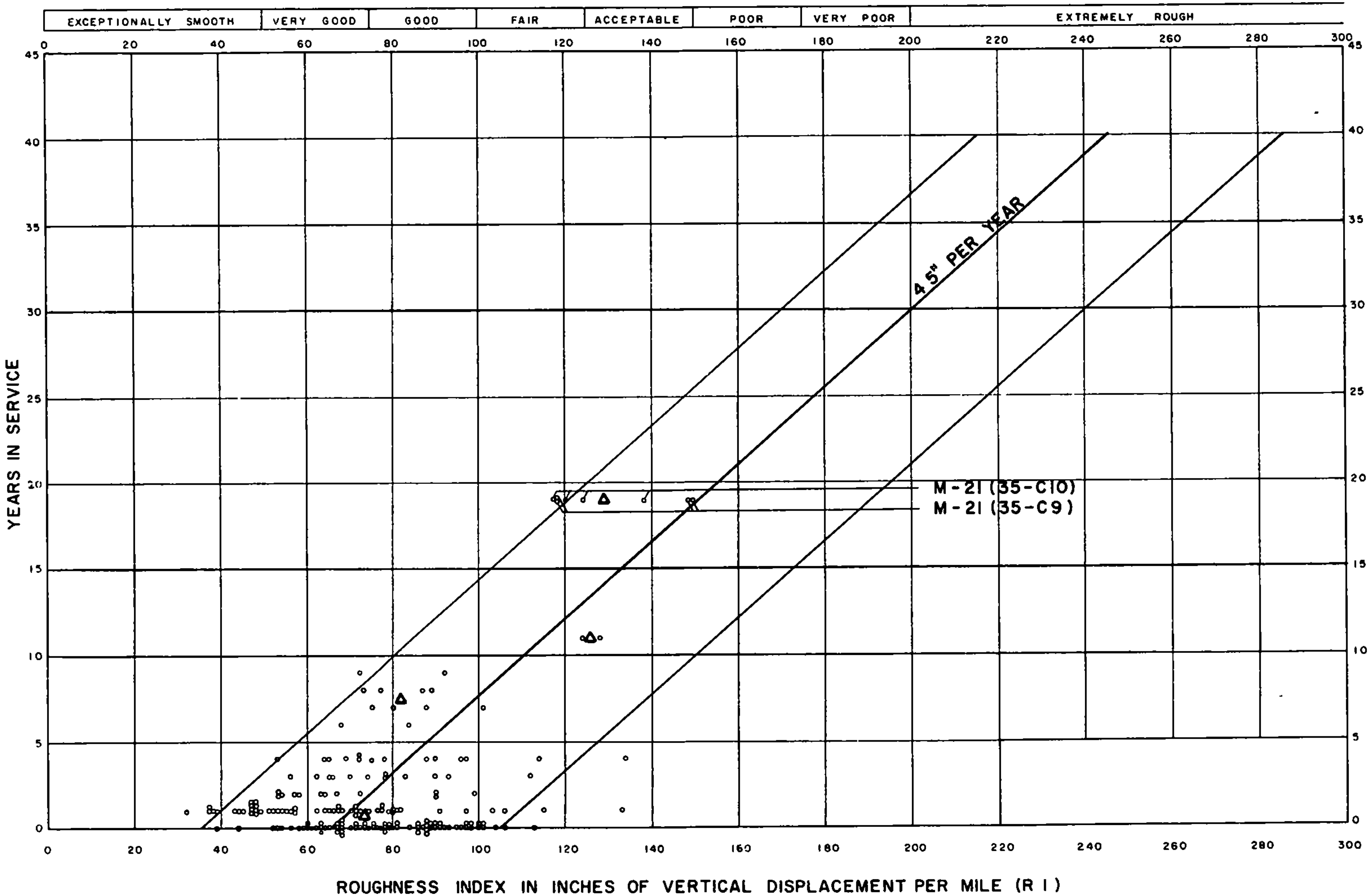


Figure 3. Roughness vs years in service, Class 1, passing lane, based on 1958-59-60 surveys.

stretch of the intersection of two major trunklines where a future grade separation was planned. Actually, it could be considered in the category of a temporary roadway for a limited service of a few years; this may have been a contributing factor in its construction.

The other three projects rougher than normal, US-23 (18-C8), US-27 (29-C3), and US-23 (4-C7), are all by different contractors. There are no special job conditions presently known to affect these results other than contractor performance as the common denominator.

## Comparison of 1959 and 1960 Roughness Indices

Although it is not considered a significant variation in pavement performance, there is a rather definite shift in Figure 1 in two groups of observations, for projects built in 1959 and 1960, which does require some explanation. It was first thought that this might be evidence of a change in built-in roughness, reflecting an accelerated construction program and possibly less effective construction control due to overload of inspection facilities and personnel. However, a more searching analysis of these observations points to the conclusion that this shift in roughness observations has been produced by a combination of factors, none of which can be demonstrated to be primarily responsible.

The 1959 projects have an average roughness index of approximately 60, somewhat less than normal, and the 1960 projects have an average roughness index of 75, somewhat above normal. Practically all of these two groups of projects were surveyed in 1960, with the 1959 projects being surveyed in the spring and early summer and the 1960 projects in the fall (after September 1) and some as late as December 15.

One factor that may have affected some of these observations is the temperature differential from June to December, with a similar but reduced differential between the top and bottom of the slab producing curling at the joints. In May and June 1960, when a number of the 1959 projects were surveyed, air temperatures ranged between 60 and 80 F. A number of the 1960 projects were surveyed in December 1960, when the air temperature ranged from 20 to 30 F, mostly in the low 20's. Unfortunately, there were no parallel surveys on the same projects in June and December which would have provided a direct comparison between the two groups of observations under discussion.

Examples of curling due to temperature differentials, however, are available from special studies where abnormal increases in roughness were reported on two recently constructed pavements of similar design. These examples are shown in Figure 2. At the bottom of the chart are pavement profiles on a short section of pavement on I-75, near Flint, where curling at the joints is the major source of the increase in roughness. This increase in roughness may include the residual deformations from two winters of cyclic change as well as some frost displacement in the subbase, which reaches a maximum about the time of the second survey in the middle of March.

A similar example is shown on the top of the chart, with the difference that the timing of the two surveys is reversed. The profile in March 1960 shows maximum roughness as the combined effect of curling and frost action. The second profile, in May 1960, shows the recovery of the pavement from the maximum temporary displacements of the seasonal cycle. Temperature differentials between the two surveys are comparable to those under discussion.

Aside from the temperature effects that may be involved, the possibility of experimental error and some effect of the accelerated construction program cannot be completely dismissed. With respect to the latter, there is no further comment except to make the obvious observation that inspection control is an ever-present problem with results in some proportion to the attention that can be devoted to it.

With reference to experimental error in the rather complex instrumentation involved in recording and integrating pavement roughness, there are always problems to be met, particularly under the requirements for mass production of pavement profiles. There were such problems during the summer of 1960, and, as a matter of fact, the profilometer was out of service for several months for a general overhaul and recalibration. Some changes in electrical circuits and mechanical details were made to improve operating characteristics and to facilitate frequent calibration. A review of the frequent calibrations during this period indicates that the data under discussion could have been

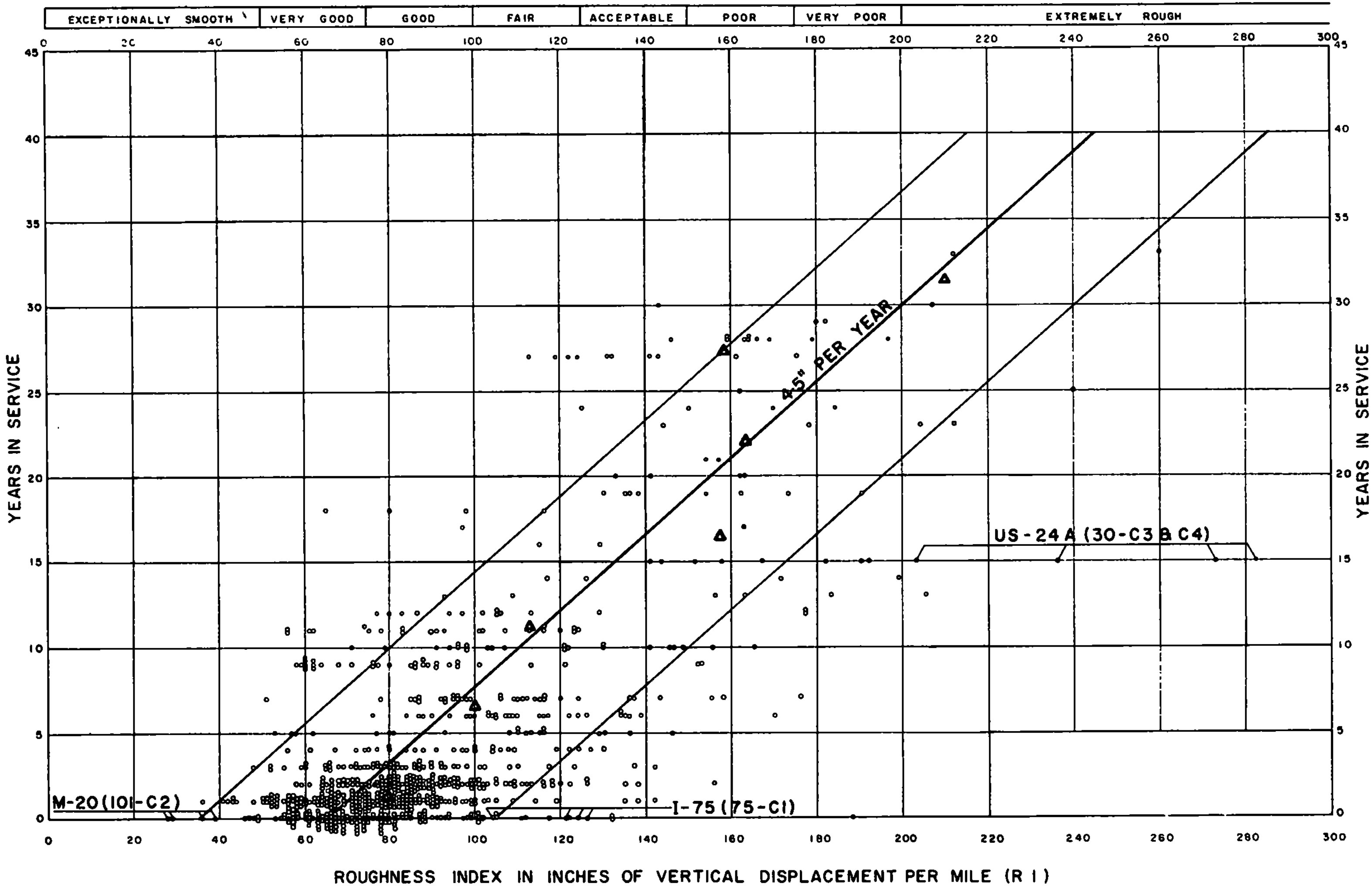


Figure 4. Roughness vs years in service, Class 2, traffic lane, based on 1958-59-60 surveys.

affected in terms of average roughness by as much as 5 percent, or a maximum of 10 percent in individual cases. This is considered to be within the normal operating accuracy of the profilometer; it is the grouping of the two sets of observations at the two time periods involved that makes such equipment error a possible contributing factor.

## Passing Lane of Class 1 Rigid Pavements

There are a number of other examples of trends in pavement performance that may be selected from the 6,000 mi of pavement profile covered by the three-year survey. Figure 3 shows the roughness indices from some 290 mi of the passing lane of dual lane divided highways. Practically all of this mileage is less than 5 years old, having been constructed as part of the Interstate system. The same band of normal pavement performance used in Figure 1 has again been used as a basis of comparison. In this case, it is apparent that the volume of data and the short period of service is not sufficient to establish any basis to differentiate between the traffic lane and passing lane. At the same time, Class 1 pavements are not expected to show any significant effect of wheel load applications, being rated as adequate or more than adequate for legal axle loads at all times. Though the available data cannot be considered conclusive, more than 90 percent of the observations in Figure 3 do fall within the band of normal behavior; the cumulative change in roughness of the limited number of older projects also follows the same trend, within the indicated limits.

For a direct comparison, two specific projects have been noted, M-21 (35-C9) and M-21 (35-C10), each with a service period of 19 years. These same projects have also been identified on Figure 1 to show that the roughness indices of the traffic lane and passing lane fall in the same range, within very narrow limits. There is one exception to this statement: a single observation of a roughness index of 225 along the outer edge of a ¼-mi section of pavement widening on this contract. A field investigation of this section is being made, but in the absence of this information it is felt that this abnormality is very probably due to a special field condition.

Again in Figure 3, there is a shift in roughness indices in the passing lane of 1959 and 1960 projects that has already been commented on in connection with Figure 1. The conditions under which these projects were surveyed and the probable contributing factors are identical with those in the traffic lane. The fact that the results are the same needs no further comment. Finally, any conclusion that could be drawn from the comparison between the traffic lane and passing lane of the Class 1 pavements would be that the available data indicate no measurable difference due to wheel load applications.

## Class 2 Rigid Pavements

The next two charts (Figs. 4 and 5) show the roughness indices on Class 2 rigid pavements. These pavements were designed to compensate for seasonal loss in strength due to subgrade deficiencies and less favorable environmental conditions. From the standpoint of load-carrying capacity, they are considered adequate for legal axle loads at all seasons of the year. In Figure 4, for the traffic lane, the data cover 244 construction contracts and some 1,275 lane-mi of pavement. The same band representing normal behavior is also shown, inasmuch as there is no evidence to support any change in these limits. Approximately 7 percent of the data show roughness indices less than normal, and 8 percent have roughness indices above normal limits.

As a whole, the data in Figure 4 are quite comparable to those in Figure 1, and show that Class 2 pavements follow the same trends in behavior exhibited by the Class 1 pavements. The cumulative change in pavement roughness with years in service shows observation points fairly well balanced around the average line or norm. The major difference in Class 2 pavements has to do with the change in Michigan design standards over the years and relates to the fact that for the last 15 or 20 years all construction on the State trunkline system has been designed and built to be adequate for legal axle loads at all times of the year. Thus, there is only a scattering of Class 2 pavements that have been in service for periods of more than 15 or 20 years. The bulk of the data in Figure

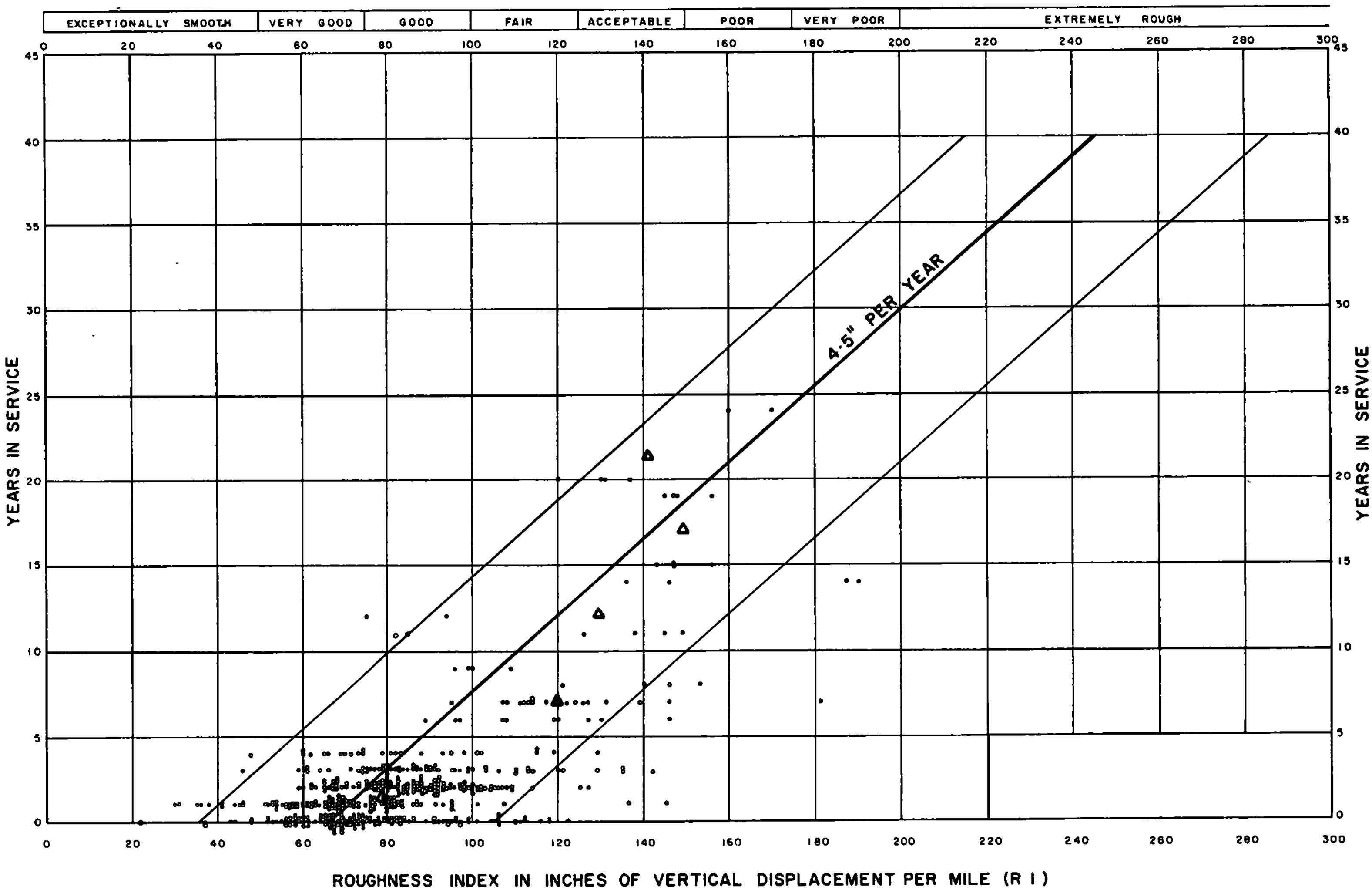


Figure 5. Roughness vs years in service, Class 2, passing lane, based on 1958-59-60 surveys.

4 is consequently concentrated in the first 15 years, with a great preponderance of the mileage having been constructed as part of the major construction programs of the past four or five years. Many of the comments made with respect to performance of the Class 1 pavements also apply to the Class 2 pavements; hence, many of the details previously discussed need not be repeated.

Next are differences in performance of the different classes of pavement as revealed by pavement profile data. The first such difference in Figure 4 is the fact that Class 2 pavements built in the first two or three years show no evidence of any shift in built-in roughness commented on in discussing the Class 1 pavements, including possible changes in construction control or inspection procedure. Inasmuch as the construction of the Class 2 pavements is coincident with the construction of Class 1 pavements, it may then be deduced that construction control was not a factor in the shift of roughness indices for Class 1 pavements. There are some significant data in Figure 4 with respect to built-in roughness, however, these being related to several special projects that have been identified. By 1959, the emphasis on pavement profiles and roughness data had been given sufficient publicity within the State, which, coupled with some competitive endeavor related to different types of pavement, stimulated contractors on certain contracts to make a special effort to build smooth pavements. Project M-20 (101-C2) is a reinforced concrete pavement on the Bay City—Midland expressway where special efforts were made to produce superior riding quality. The fact that the average roughness index on all four lanes of this project was less than 40 is an indication of what can be done with respect to built-in roughness when sufficient effort is applied. Another project shown in Figure 4, I-75 (75-C1), built in 1958 and surveyed the same year, represents the top of the range in initial roughness. Construction reports from the project revealed that poor subbase compaction left the paving contractor with inadequate support for the forms. This difficulty was reported at the time of construction as affecting the quality of the work.

Although all the projects in Figure 4 that showed abnormal performance are being investigated, only one other example has been identified for discussion in this report—Project US-24A (30-C3 and C4). This project stood out because of its poor riding quality in the earliest days of the pavement performance study. Therefore, it received immediate attention and has, as a matter of fact, been commented on in previous reports (4). This contract shows the result of using short, 20-ft slabs of plain concrete without load transfer at the joints in an effort to control pavement cracking. The subgrade was a heavy lake-bed clay with a fill of several feet produced by side-casting from the ditches. This fill was allowed to weather for two years before the pavement was constructed; then, an 18- to 24-in. sand subbase was added to provide more adequate subgrade support and to eliminate pumping action. The crack control was successful in that very few of the 20-ft slabs have cracked, but this design produced one of the roughest riding pavements in southern Michigan due to tilting of the slabs and faulting at the joints.

## Passing Lane of Class 2 Rigid Pavements

In Figure 5, roughness indices for the passing lane of Class 2 rigid pavements have been assembled. These data cover 139 construction contracts and 277 lane miles of pavement. There are only a few projects in the survey more than 10 years old, for reasons already cited, and a high proportion of the roughness data are from projects built in the last 5 years. More than 90 percent of the data fall within the same band of normal behavior, with 2 percent of the points indicating roughness indices less than normal, and 6 percent greater than normal. There is no evidence from these limited data of any differential in the cumulative change in roughness between the passing lane and traffic lane of Class 2 pavements.

There are fewer projects showing evidence of abnormal behavior outside the band of normal performance and these are being investigated for special conditions that may have produced these results. With respect to built-in roughness, there are several projects that are exceptionally smooth and several that are rougher than normal. These include the passing lane of projects already cited in connection with Figure 4 for the traffic lane of Class 2 pavements, so no further comment is required.

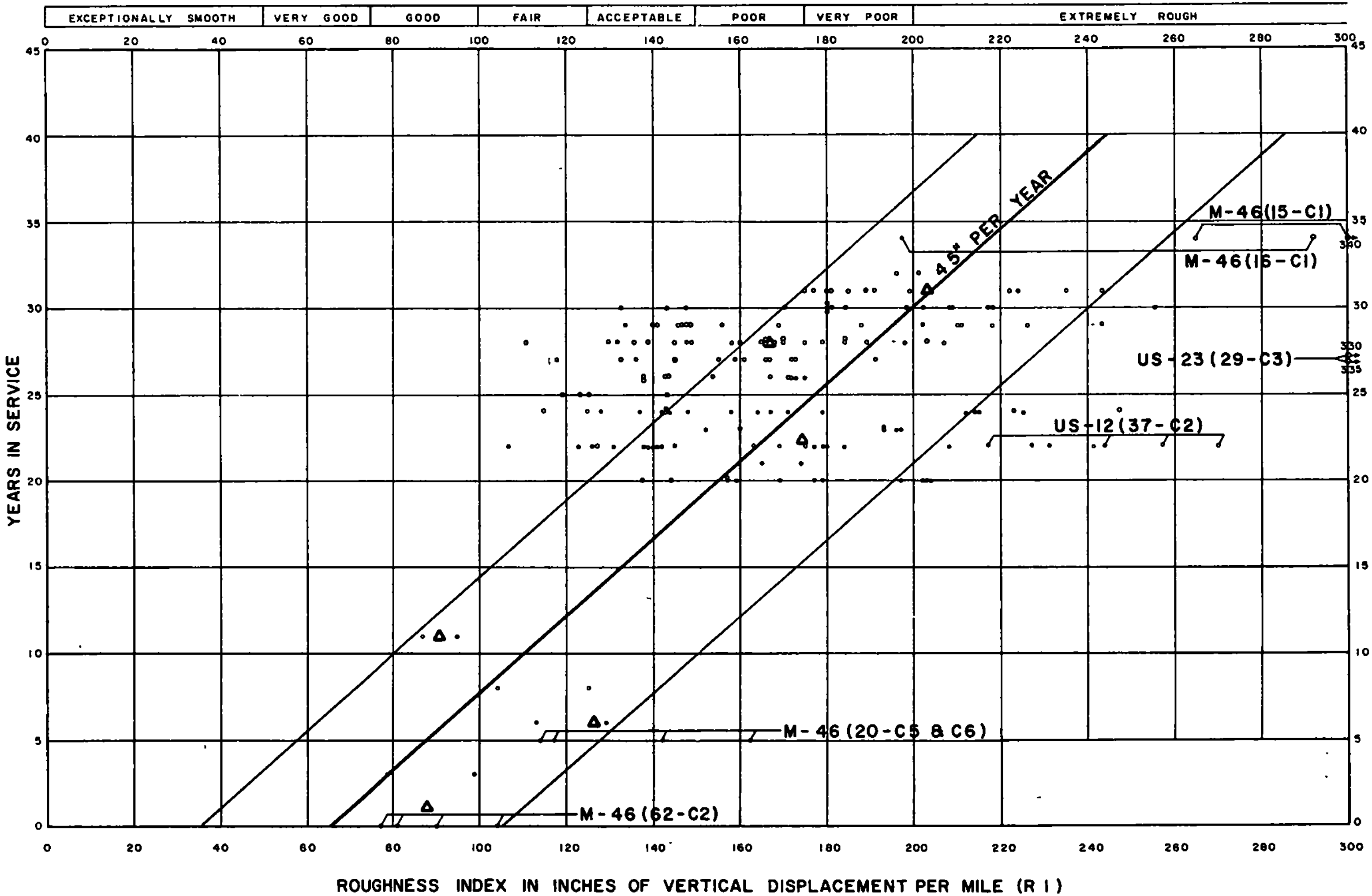


Figure 6. Roughness vs years in service, Class 3, traffic lane, based on 1958-59-60 surveys.

## Class 3 Rigid Pavements

The performance of Class 3 rigid pavements, insofar as they are covered in the three-year survey, is shown in Figure 6. The data are from 70 construction projects and 353 lane-mi of pavement. Most of the projects are more than 20 years old and represent outdated design and construction standards which do not compensate for seasonal loss of strength, and may therefore be regarded as deficient for legal axle loads, at least during the spring breakup.

There are a few projects less than 15 years old still rated as Class 3 pavements, a fact that appears to be inconsistent with present Michigan design standards. Further investigation of the two projects identified in Figure 6 determined that they have been rebuilt to present standards and should now be reclassified, but this correction had not been made at the time of the profile survey. M-46 is an east-west trunkline across the State, running west from Saginaw and carrying fairly heavy truck traffic generated by the petroleum industry in this area. The other two projects identified were 34 years old at the time of the survey in 1959 and had become extremely rough, requiring heavy maintenance to keep them in service. These two sections have subsequently been strengthened and resurfaced, but have not yet been resurveyed. These recent construction contracts have been betterment projects consisting of overlays to reinforce the present pavement without complete rebuilding to correct the basic deficiency in subgrade support and inferior drainage, the results of which are still considered debatable. Although the pressure to bring the rest of this road up to present day standards has been heavy, the cost of rebuilding, involving complete relocation, has postponed its programming in competition with other important routes also requiring attention.

Further discussion of the performance of Class 3 pavements would point to the much wider range in performance of those projects more than 20 years old. This wide variation in behavior, as compared to what has been selected as normal performance, suggests less attention in design and construction to those factors most responsible for pavement performance; namely, soil conditions, drainage, and environment. In this connection, it is noted that 25 percent of the roughness indices are less than normal and 14 percent are greater than normal.

It is probable that further investigation of projects showing abnormal performance will lead to reclassification of a number of the projects on both sides of the band of normal performance. It is almost certain that those projects which have become extremely rough would require complete reconstruction or extensive correction of those deficiencies that have lead to poor performance to bring them up to present day standards. One project in this group, US-23 (29-C3), can be used for illustration. This project, which was 27 years old when surveyed in 1959, had roughness indices of 330 and 335 in. per mi and is built over a complex of poorly drained heavy clays and silty sands. It has carried heavy traffic in later years and has required heavy maintenance to keep it in service. It was resurfaced in 1959, shortly after the profile survey, and has now been replaced by a modern expressway, I-75 of the Interstate system. The other project identified, US-12 (37-C2) was built in 1937 and was extremely rough when surveyed in 1959 after 22 years of service under heavy traffic. It, too, was built over poorly drained soils of heavy clay and some areas of muck and swamp-border soils in low-lying areas. The concrete pavement was reinforced and of standard thickness, but lacked the granular subbase now used to compensate for subgrade deficiency and to eliminate pumping. It was resurfaced shortly after the profile survey and has now been replaced in the trunkline system by a modern expressway, I-94 of the Interstate system.

## Class 4 Rigid Pavements

A few Class 4 pavements were included in the profile surveys and are shown in Figure 7 more to complete the picture of pavement performance than to provide design data of current interest. There are only 7 contracts and 20 lane-mi of pavement, all of which are more than 30 years old. All of these pavements are rated as extremely rough and fall in the upper range or above the band of normal behavior which has been shown for comparison. This performance is consistent with the Class 4 rating, indicating pavements inadequate for legal axle loads at all times. Special field investigation of these

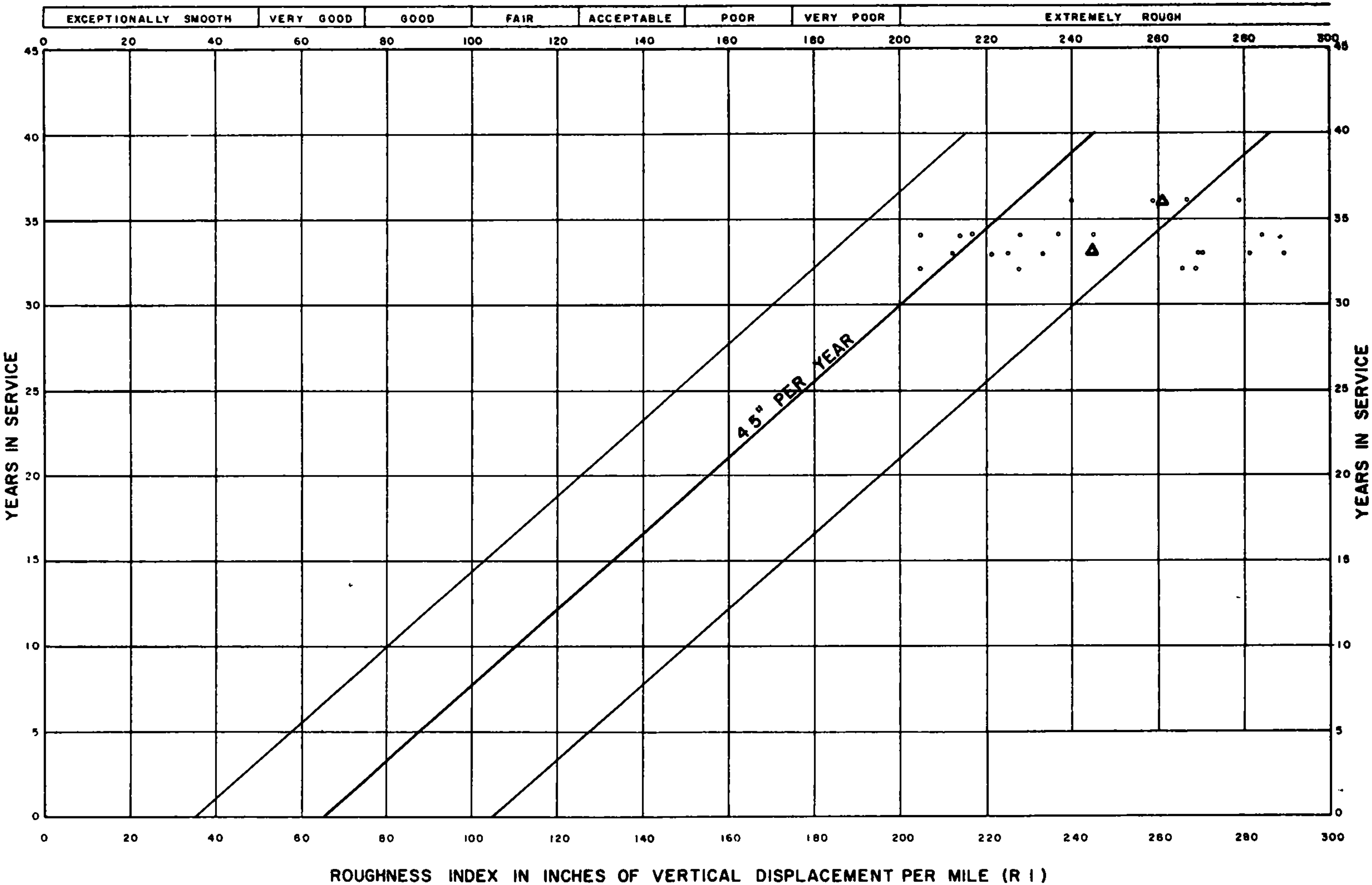


Figure 7. Roughness vs years in service, Class 4, traffic lane, based on 1958-59-60 surveys.

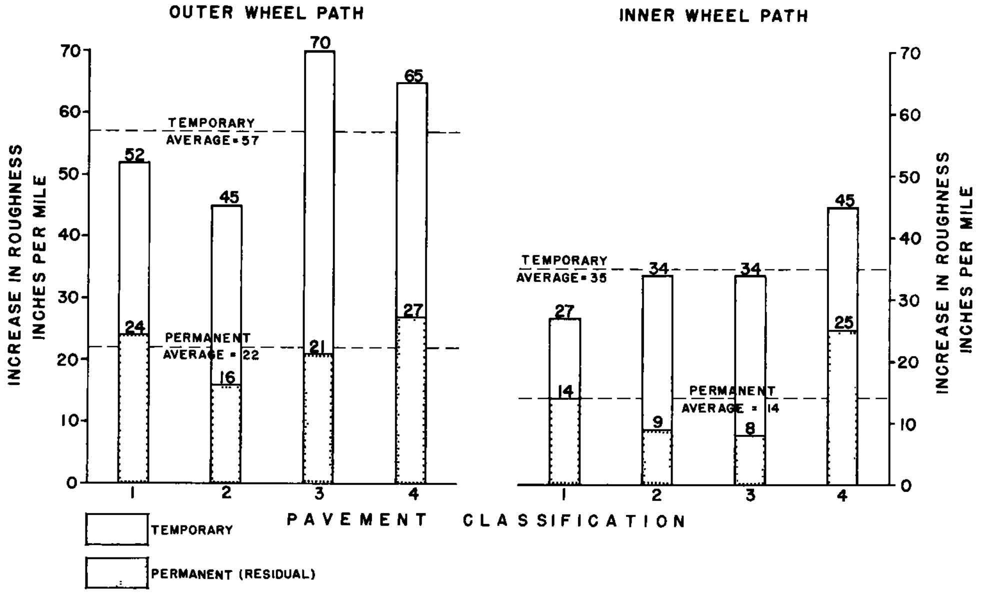


Figure 8. Seasonal fluctuation in pavement profile.

projects has not been made and may not be required as they can be identified with a history of heavy maintenance and substandard conditions.

## SOURCES OF PAVEMENT ROUGHNESS

The foregoing review of cumulative changes in rigid pavement with age in service has been based almost entirely on an analysis of 6,000 mi of pavement profiles. It may be desirable to supplement these data by some discussion of special studies which may serve to emphasize several important factors contributing to pavement roughness. These factors may be briefly described as temporary and permanent roughness due to frost displacement, built-in roughness, and deflection due to wheel load applications.

### Frost Displacement

In the first two years of the Michigan pavement performance study, a number of special pavement sections were selected throughout the State to measure the seasonal fluctuation in pavement profiles due to frost action. Some of the data have been previously reported and will be only briefly presented here (6). Figure 2 showed several pavement profiles illustrating temporary displacement due to curling at the joints combined with frost displacement in the granular subbase. As shown by a comparison of pavement profiles at different times of the year, much of this displacement at the time of maximum frost action was temporary and disappeared in the summer profile. However, there was some residual or permanent displacement remaining after each cycle to which the pavement was subjected.

Figure 8 shows a seasonal fluctuation in pavement profiles through the rather severe winter of 1959 and the succeeding summer. The frost displacement is shown for four classifications of pavement in terms of the average increase in roughness in inches per mile for all pavement sections in each class. The shaded areas represent the permanent or residual roughness contributing to the cumulative increase in roughness over a period of years. There was no consistent correlation between frost displacement and pavement classification from the standpoint of structural adequacy, although there was some trend in that respect.

This type of frost displacement is to be distinguished from the deep-seated frost-heaving which was a problem in former years, but which has now been largely eliminated

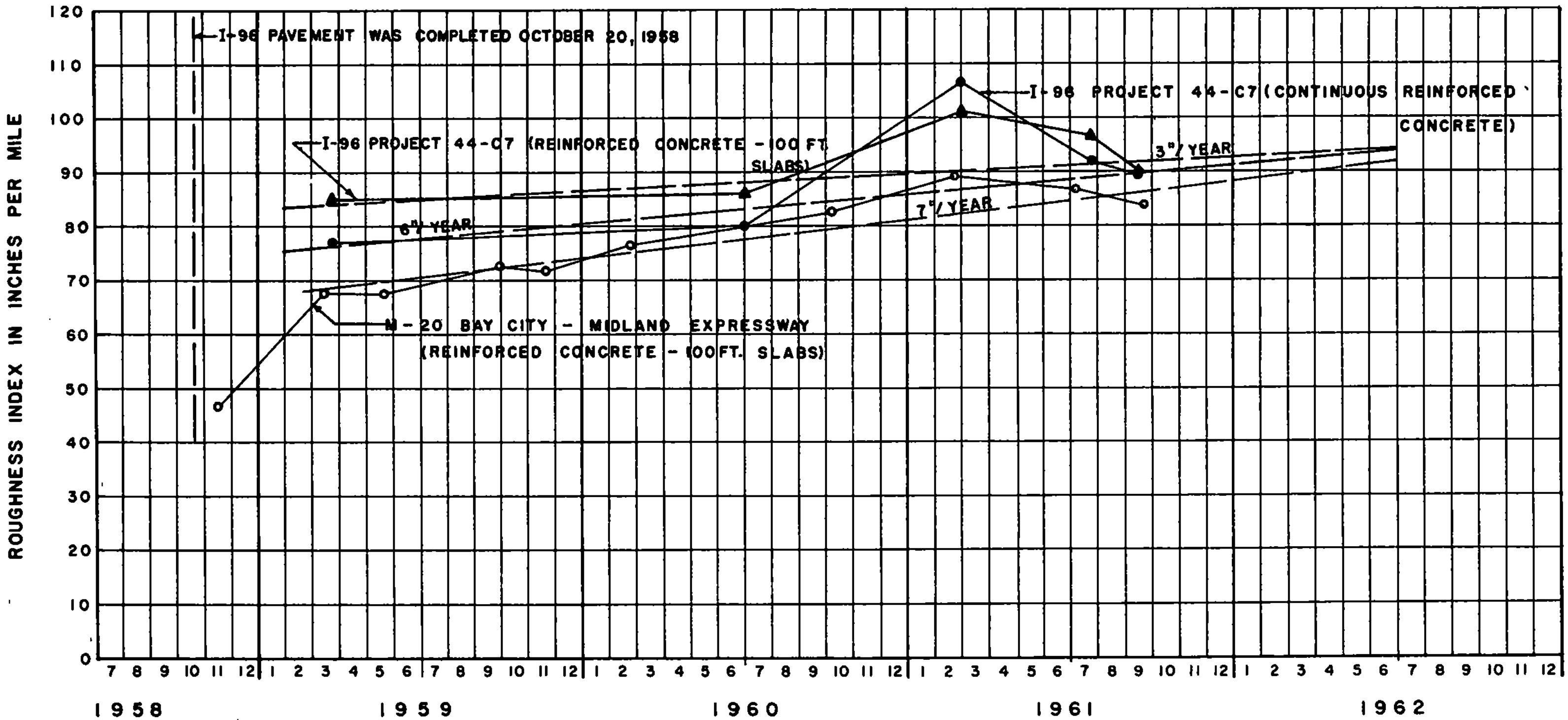


Figure 9. Cumulative change in roughness on specific projects.

from State trunklines. Such frost displacement is associated with freezing of moisture accumulating in the granular bases and subbases through infiltration at the shoulder or through joints and cracks, or condensation under the surface of moisture entering in the vapor phase. Thus, the highest classification of pavements, from the standpoint of load-carrying capacity, were affected, as well as the less adequate roads.

From the standpoint of cumulative change, Figure 9 shows several specific projects for which profile data are available over several successive years, illustrating the rate of increase in roughness the first few years after construction. There are both Class 1 and Class 2 rigid pavements in this example, and data are shown from profiles of an experimental section of continuously reinforced concrete pavement on I-96. The rates of increase in roughness as shown are an approximate average, excluding the first winter, as the residual roughness for the initial cycle of displacement does not appear to be representative. There is also considerable variation depending on the severity of winter from the standpoint of frost action. In general the rate of increase, varying from 3 to 7 in. per mi per year in these several pavements in their early years of service, may be expected to level off over a considerably longer period of service.

## Built-in Roughness

Figure 10 shows one source of built-in roughness that relates to pavement finishing in which the cause and effect is so specific that it needs no particular comment. The excessive roughness of bridge decks and short lengths of pavement slab where hand-finishing is employed has been a problem of much concern to the Michigan State Highway Department for some time. Figure 10 shows direct comparison between sections of pavement or bridge deck, as the case may be, where hand-finishing has been employed and adjacent pavement which has been machine finished. Transverse finishing of bridge decks is one of the possibilities under investigation and it has shown some promise.

## Roughness Due to Wheel Load Applications

Pavement deflection under wheel load applications and the permanent displacements caused by excessive load are a source of loss in riding quality that has always been of major concern to highway engineers. In the roughness data from some 6,000 mi of pavement profile reviewed in this report, two important facts stand out, qualified necessarily by the limitations in the volume of supporting data available. First, there were measurable increases in the cumulative roughness over a period of years in the Class 3 and Class 4 rigid pavements that were known to be deficient in load-carrying capacity. This deficiency was emphasized in specific projects that were cited where their poor performance had been more definitely related to known deficiency in subgrade support.

Second, there was no measurable difference in the cumulative roughness of the traffic lane and passing lane of Class 1 and Class 2 pavements in the periods of service covered by the profile data presented. The necessary qualification in this statement is in recognition of the short period of service which is related to two developments in highway construction in Michigan. First, the construction of dual-lane divided highways, where such a comparison could be made, has largely taken place in the last 10 years. Second, it is only in the last 15 or 20 years that Michigan design standards have required that all trunkline construction be made adequate to carry legal axle loads at all times of the year.

These qualifications notwithstanding, the evidence accumulated is entirely consistent with the statement made by Commissioner Curtiss of the Bureau of Public Roads in discussing damage caused by highway loads when he said, in effect, that properly built roads would not be damaged by the loads for which they were designed. Evidence bearing on the same point is provided by pavement profile data from Class 1 and Class 2 pavements showing that the cumulative increase in roughness with years in service is closely related to soil conditions, drainage, and climatic environment. This in turn illustrates the point made by the late Commissioner MacDonald of the Bureau of Public Roads when he said: "The roads are more destroyed really by climatic and soil conditions than they are by any use that is made of them."

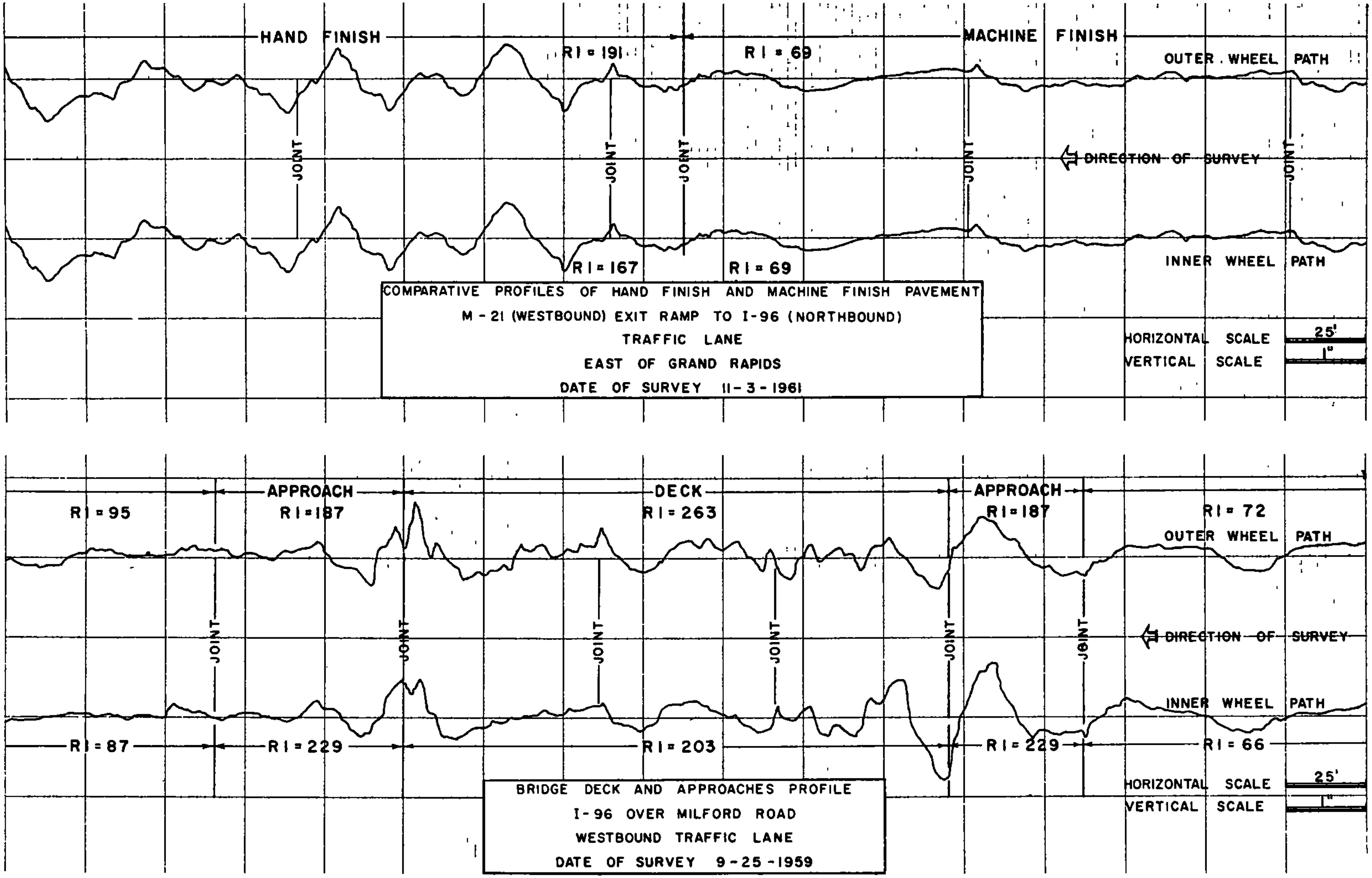


Figure 10. Comparative profiles.

## CONCLUSIONS

The Michigan pavement performance study has now been in progress for four years and has accumulated some 8,100 mi of pavement profiles, including those from the 1961 surveys. This paper presents a substantial part of the data on rigid pavements for the first three years, covering 966 construction contracts and some 6,000 lane-mi of pavement in all parts of the State. The analysis of these data is still in progress; it is felt that additional information of value in pavement design and construction is yet to be obtained from the data, particularly in the study of all those projects whose performance varies considerably from normal behavior.

This study has been predicated on the belief that accurate recording of the condition of existing pavements after years of service under actual field conditions and an objective analysis of the resulting data was a most promising approach for evaluating pavement performance and relating it to pavement design and construction. In more specific terms, the procedure involved (a) accurate recording of the profile of the pavement surface which has been evaluated quantitatively in terms of a roughness index in inches of vertical displacement per mile, supplemented by a study of the magnitude and character of these displacements as revealed by the recorded pavement profile; and (b) the cracking of the pavement recorded and evaluated quantitatively as a continuity ratio directly related to the average interval between cracks and joints.

From these data the following tentative conclusions, subject to further study and possible revision, summarize the results:

1. Pavement performance and cumulative change in rigid pavements, under Michigan environment and service conditions, can be expressed in terms of a band of normal performance which brackets approximately 85 percent of the data, excluding those projects showing abnormal performance.
2. The width of this band of normal performance has been taken as 70 in. of vertical displacement per mile as representative of the limits within which variations in riding quality have been and perhaps can be controlled by design and construction practices.
3. Initial or built-in roughness, in terms of inches of vertical displacement per mile, ranges from 35, which is considered exceptionally smooth, to 105, rated as only fair. From a general consideration of this range and from results on specific projects cited, this is one factor in riding quality that appears to offer an obvious opportunity for substantial improvement in the upper limit and any variation above this limit.
4. The progressive increase in pavement roughness, or cumulative change with age in service, which has been characterized as normal, averages 4.5 in. per yr and has been based largely on the performance of Class 1 and Class 2 pavements designed and built to carry legal axle loads at all times of the year under Michigan climatic environment.
5. Both comparison of the traffic lane and passing lane of Class 1 and Class 2 pavements, within the limits of available data, and analysis of specific projects showing abnormal behavior indicate that an average cumulative increase in roughness of a magnitude of 4 to 5 in. per mi per yr results from the characteristics of rigid pavements subjected to Michigan climatic conditions. This finding is confirmed by special studies of frost displacement and effect of temperature differentials and appears to identify this range as one that may not be effectively controlled under present design and construction practices.
6. The data from Class 3 and Class 4 pavements, recognized as deficient in load-carrying capacity, do show evidence of an added increase in roughness due to wheel load applications. These data come from older roads not representative of present design standards in Michigan and consequently provide a limited basis for drawing specific conclusions. However, when supplemented with more definitive results from specific projects showing abnormal behavior and viewed in the light of favorable results from Class 1 and Class 2 pavements, there is clear indication that deficiency in subgrade support is the primary factor in their inferior performance.
7. Finally, with respect to procedure for measuring pavement performance and relating it to design and construction conditions, it is found that the projects showing abnormal behavior provide the most significant information. Analysis of all of these proj-

ects has not been completed, but examples that have been presented in this report provide evidence that the investigation of existing pavements under actual service conditions and environment fulfills the promise it was felt to hold for a realistic appraisal of pavement design and performance.

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# Effect of Pavement Condition on Dynamic Vehicle Reactions

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A vehicle traveling over a highway containing surface irregularities experiences vertical motion as well as horizontal motion. Associated with the vertical motion are forces between the highway and the vehicle that are developed in addition to the static weight of the vehicle. These forces are frequently referred to as the dynamic reactions or the dynamic forces. They depend on the vehicle suspension characteristics, the condition of the pavement, and the velocity of the vehicle.

Laboratory tests were conducted to determine the frequencies of vibration at which passenger vehicles would develop large forces between the tires and the pavement. The relationship between force exerted by the tire and vertical displacement of the tire tread was measured at all frequencies at which any appreciable force was developed. The vehicle characteristic thus obtained included the actual effects of all components of the vehicle suspension system.

A criterion of pavement condition was established by making power spectral density analyses of highway elevation measurements. A brief description of the physical significance of this criterion is included.

A procedure is discussed for combining the vehicle characteristics with an elevation power spectrum (pavement condition criterion) at a selected vehicle velocity to obtain a dynamic force power spectrum. The usefulness of this result is discussed and curves of the root-mean squared value of the dynamic force vs vehicle velocity are included for three different pavement conditions.

By using the criterion of pavement condition as defined in this paper it is possible to estimate the dynamic force that one wheel of a vehicle will exert on a highway.

• A VEHICLE parked on a smooth, level highway will exert a force on the pavement that is equal to the combined weight of the vehicle and the cargo. This force is frequently referred to as the static force. If the vehicle moves at a constant velocity over the highway there will be no increase in this force if the tires of the vehicle are properly balanced.

If, however, the vehicle moves along a highway containing pavement irregularities, the vehicle will also experience vertical motion. This motion will be very abrupt if sudden discontinuities such as chuckholes or faults are present. The larger the highway discontinuities, the more violent will be the resulting vertical motion. If large vertical displacements are accompanied by large accelerations, it is evident that the vehicle must experience large forces that will have their origin in the highway. These forces will be in excess of the static force and will be referred to as the dynamic forces.

Although large discontinuities may produce large vertical accelerations and hence generate large forces between the vehicle and the highway, it is possible to reduce the magnitude of these forces by reducing the velocity of the vehicle. Thus, if the vehicle travels slowly enough over a very rough highway, the increased force between the vehicle and the highway may be held within desired limits.

Large vertical accelerations can also result from smooth undulations in the highway. If the speed of a vehicle is increased sufficiently, a variation in the highway profile that may be unnoticed at a slower speed may result in a violent pitching of the vehicle at a higher speed. It is thus possible for the vehicle to experience large vertical forces from conditions other than discontinuities in the pavement.

Also, characteristics of the vehicle suspension system will influence the behavior of the vehicle. A vehicle with very hard tires will react much more violently to an irregularity than will a vehicle with soft tires. Moreover, the springing between the body of the vehicle and the axles will influence the response of the vehicle on the highway. It is thus evident that the dynamic reaction between a vehicle and a highway will be influenced by the condition of the pavement, the vehicle suspension characteristics and the velocity with which the vehicle is operated.

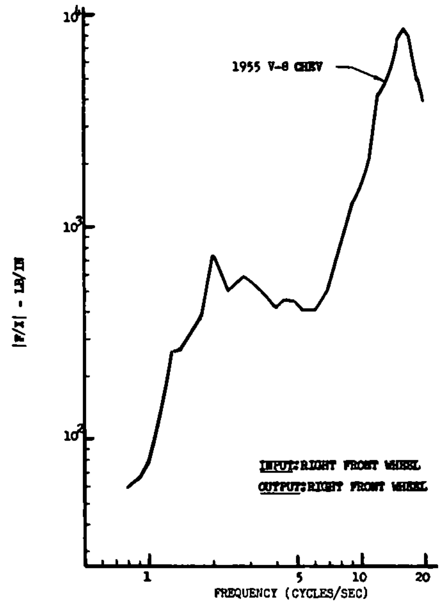


Figure 1. Maximum envelope vehicle response characteristics.

## DETERMINING SIGNIFICANT WAVE LENGTHS IN PAVEMENTS

A highway profile contains a tremendous range of wave lengths; close examination reveals very small irregularities in the surface due to finishing. These may have a wave length in the order of a fraction of an inch and a correspondingly small amplitude. On the other hand, the highway may go over hills and down valleys containing wave lengths several miles in length. Between these extremes are undulations having intermediate wave lengths. It is, therefore, necessary to determine the wave lengths in the profile that will be significant in influencing the dynamic force that the vehicle exerts on the pavement.

To determine the significant wave lengths, it is necessary to determine the significant frequencies of vibration of the vehicle. To do this, it is convenient to consider the ratio of the force F which the vehicle exerts on the highway to the vertical displacement X of the tread of the tire. This can be done by constructing a platform that will move up and down with simple harmonic motion having a controlled frequency and an adjustable amplitude X. An electronic scale can be placed on the platform to measure the resulting reaction F. If a wheel of a vehicle is driven onto the platform and the platform is excited, the ratio of F to X can be obtained (1).

A typical vehicle characteristic is shown in Figure 1. It is evident that the vehicle responds to a much greater extent to certain frequencies of vibration than to others. When the wheel is excited at frequencies below 1 cycle per sec (cps), very little force is developed in excess of the static force. At 2 cps a relatively large force is generated, and as the frequency is increased, this force decreases slightly. At 7 cps the force again increases and reaches its maximum value at 15 cps. With a further increase in frequency the force decreases rapidly, and no further increase in force has been observed up to 20 cps. A curve of this type obtained experimentally thus describes the suspension characteristics of a vehicle and includes the effect of tires, springs, shock absorbers, and all other components of the vehicle.

Variations in the highway profile that will excite frequencies of 2 or 15 cps will thus generate large forces between the vehicle and the highway. These frequencies are therefore of interest in predicting the influence of the pavement on the reaction of the vehicle.

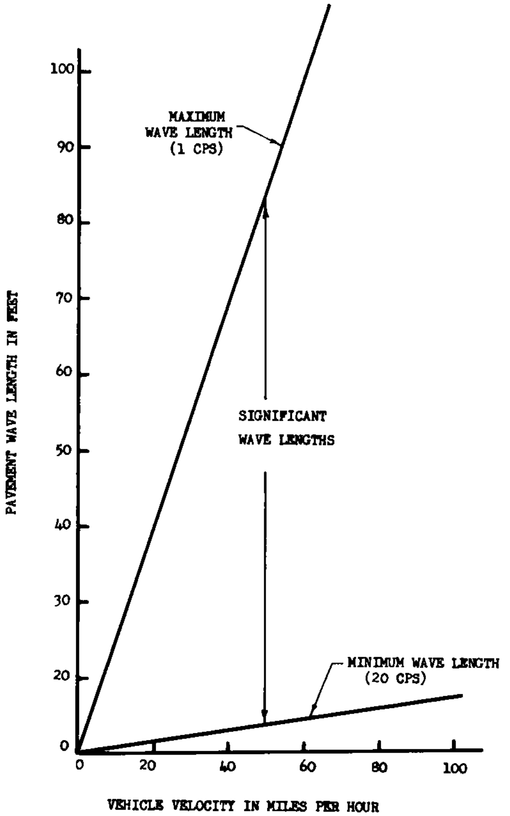


Figure 2. Pavement wave lengths exciting frequencies from 1 to 20 cps in vehicle.

The velocity of a vehicle will influence the frequency of excitation that is received from the highway. For example, a certain highway profile is an endless succession of sine waves each having a wave length of 5 ft. If the vehicle described in Figure 1 travels over this highway at 10 ft per sec it will pass over two complete sine waves per second. Excitation of the vehicle at 2 cps will thus result. If the velocity is increased to 75 ft per sec the vehicle will pass over 15 complete sine waves in 1 sec thus experiencing a 15 cps excitation. A fixed wave length can therefore induce any exciting frequency in the vehicle if the appropriate vehicle velocity is selected.

From Figure 1, it is evident that the range of frequencies causing large dynamic forces between the highway and the vehicle extends from 1 to approximately 20 cps. Wave lengths that excite frequencies of less than 1 cps will cause no appreciable increase in the dynamic force, and wave lengths that excite frequencies appreciably greater than 20 cps will also make very little contribution to this reaction. Thus, only frequencies lying between 1 and 20 cps are of much interest in predicting the dynamic force. A curve showing the significant wave lengths vs vehicle velocity is given in Figure 2. In this figure, the longest wave length of significance (which would generate a disturbance at 1 cps) and the shortest wave length (which would induce a disturbance of 20 cps) are shown by the two lines. At any given velocity the vertical distance between the two lines represents the range of wave lengths that are significant in inducing appreciable forces between the vehicle and the highway. From Figure 2 it is evident that the shortest wave length at 50 mph is of no interest at 80 mph, whereas the long wave length at 50 mph is still effective in producing dynamic force when the vehicle is traveling at 80 mph. It is also evident from Figure 2 that higher vehicle velocities introduce longer wave lengths into the response of the vehicle while eliminating the shorter wave lengths. Having the information shown in Figure 1 for any vehicle would therefore define the region of interest in Figure 2.

## DESCRIPTION OF PAVEMENT CONDITION

A suitable criterion is needed to describe the condition of the pavement. This criterion should be based on measurements and should be of such a nature as to be readily used with the vehicle characteristic shown in Figure 1.

A statistical procedure (2) is available for describing random processes, and it is interesting to consider the possible use of this technique for describing pavement conditions. Although a pavement does not completely satisfy all of the requirements, it comes reasonably close in many respects. In the process of applying this procedure it is possible to check the extent to which the pavement fulfills an important necessary condition, and the analysis can be discontinued if the pavement does not display the proper characteristics.

The procedure under consideration is that of making a power spectral density analysis. Because many references (3, 4) are available that give detailed discussions of this technique no attempt will be made to present these details in this paper. The important considerations in applying the procedure to highway problems will be presented, however.

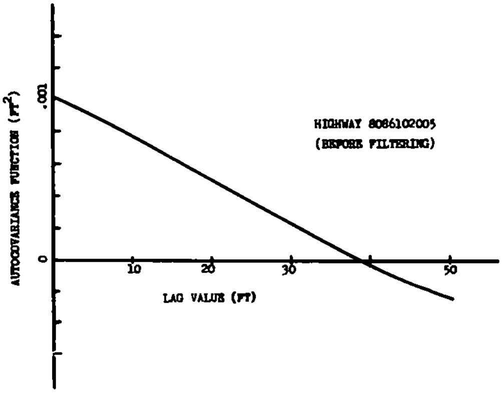


Figure 3. Typical pavement autocovariance function.

Although this analysis can be applied to any type of measurement, it is instructive to apply it to highway elevation measurements made at 1-ft intervals using a rod and level. The spacing of these elevations is arbitrary and depends on the wave lengths that are to be resolved. Elevation measurements will not attenuate the long wave lengths that become increasingly important at higher vehicle velocities, and when made with this spacing include wavelengths down to 2 ft.

An initial data processing operation is necessary to obtain the difference between each elevation measurement and a properly selected base line. From the differences thus obtained the autocovariance function is computed. This can most easily be done on a digital computer, and a typical function for a highway is shown in Figure 3. This function should approach zero and remain at zero as the lag values increase, and the function should not be periodic. The extent to which the autocovariance function fulfills these conditions determines whether the pavement profile is sufficiently random to be described by this technique. An unsatisfactory autocovariance function can frequently be improved by performing a filtering operation on the original data to remove existing periodicity. Errors in the original data can usually be detected from this function. Having obtained the autocovariance function, it is then possible to compute the power spectrum.

To understand the physical significance of a power spectrum analysis of highway elevation measurements, it is important to realize that the power spectrum is computed from the variations in these measurements. If the highway is smooth and level so that the elevation measurements are all the same, all values for the power spectrum will be zero. It is thus evident that only variations in elevations spaced 1 ft apart will be obtained in this analysis. If between two successive elevation measurements a pavement surface should rise sharply and then return to its original elevation, the power spectrum would not be aware of this condition. At first thought, this may appear to be a very severe limitation on the accuracy of a power spectrum based on measurements made at 1-ft intervals to describe pavement conditions, as it is evident that there are irregularities in the highway profile that are smaller than 1 ft in length. Figure 2 shows, however, that at any appreciable velocity the very short wave lengths will have very little effect on the behavior of the vehicle. Thus, a pavement irregularity smaller than 1 ft in length would have to be of appreciable amplitude in order to have a discernable effect on the vehicle. It should thus be realized that the power spectrum contained in this paper can only account for pavement irregularities that would be detected by making elevation measurements at 1-ft intervals.

The variation in the elevation measurements (a measure of pavement roughness) is most easily measured in terms of feet squared, and the ordinates of the power spectrum curve are in terms of feet squared per cycle per foot. The abscissae are in units of feet per cycle, which is the reciprocal of the wave length. A highway power spectrum is thus a purely geometric quantity and does not contain any units of time. When the power spectrum is plotted, the total area under the power spectrum curve will represent the mean squared value of the variation in the elevation measurements, and this value can also be used as a criterion of pavement condition. Of greater importance is the fact that the area under the power spectrum curve between any two selected wave lengths will indicate the contribution to the total roughness or variation in the highway elevation measurements that this range of wave lengths produces. Thus, from the

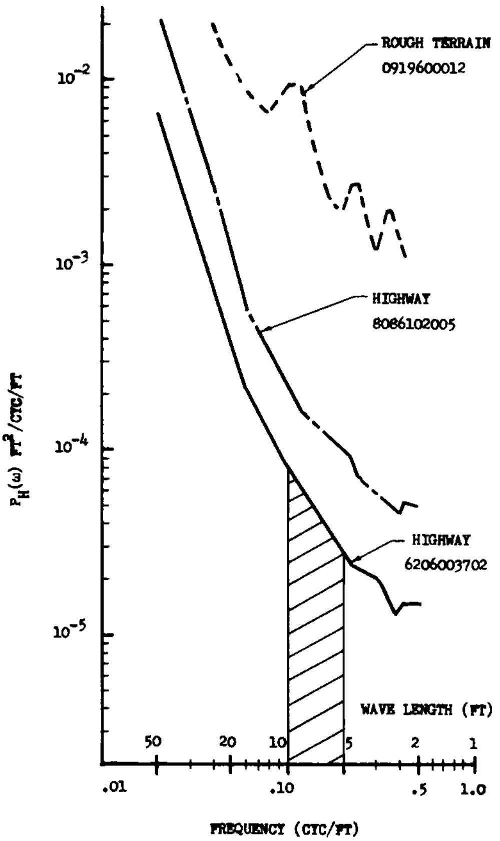


Figure 4. Highway power spectra.

power spectrum curve, it is possible to determine the wave lengths in the pavement that make the greatest contribution to the variation in the elevation measurements.

Typical power spectra curves are shown in Figure 4. The lowest curve in this figure is the power spectrum of an extremely smooth highway. This is to be expected because the area under this curve is less than the area under the other curves shown in the figure. The dotted curve shown at the top of the figure is the power spectrum for rough terrain comparable to a cow pasture. Two extremes are thus represented in Figure 4. Virtually all highways could be expected to lie between these two curves. A highway in fair condition is represented by the middle curve in Figure 4.

Along the horizontal axis is a special scale showing values of wave length. The wave length increases to large values moving from right to left along the horizontal axis. These curves indicate that the greatest variation in the elevation measurements is due to long wave lengths present in the highway. It can thus be seen that large variations are associated with long wave lengths and relatively small variations are associated with short wave lengths. The shaded area lying between wave lengths from 5 to 10 ft long represents the amount of irregularity or roughness that these wave lengths contribute to the total variation in the highway profile. Because at higher vehicle velocities the longer wave lengths become more significant as shown in Figure 2, it is a little disturbing to note that the longer wave lengths tend to increase the excitation to which the vehicle is subjected.

A highway elevation power spectrum can be used with the vehicle characteristics shown in Figure 1, if the velocity of the vehicle is introduced. This is done by multiplying the abscissae of the power spectrum curve by the vehicle velocity to obtain the frequency in cycles per second, and by dividing the ordinates of the power spectrum curve by the vehicle velocity to obtain the units of feet squared per cycle per second. A different curve will thus result for each velocity. By making this transformation, it is now possible to plot the power spectrum curve with the vehicle characteristic shown in Figure 1. This is done in Figure 5 where the vertical axis on the left side indicates the F/X ratio of the vehicle while the vertical axis on the right side indicates the power spectrum values for the highway. This has been done for two different vehicle velocities, and it is evident that as the velocity of the vehicle increases, the longer wave lengths make more of a contribution to the total excitation of the vehicle. Having the information shown in Figure 5, it is now possible to compute the dynamic reaction of the vehicle on the highway.

## COMPUTING THE DYNAMIC FORCE

Another property makes these characteristics for the vehicle and the highway very useful. It is possible to combine them to obtain an estimate of the dynamic force that the vehicle will exert on the highway. To do this, it is first necessary to modify the highway characteristic by introducing the vehicle velocity as previously described. An additional modification must also be made to convert the units of the highway power spectrum from feet squared to inches squared. If this is done, the relationship (5, p. 197) that can be used to determine a power spectrum of the dynamic force that the vehicle will exert on the highway is

$$P_F(f) = P_H(f) \mid F/X\,(f) \mid^2$$

in which

$P_F(f)$ = power spectrum of the dynamic force as a function of frequency in cycles per second (units: $lb^2/cps$);

$P_H(f)$ = power spectrum of the deviations of the pavement elevations as a function of frequency in cycles per second (vehicle velocity included, units: $in.^2/cps$); and

$F/X(f)$ = ratio of dynamic force to tire tread displacement (vehicle characteristic) as a function of frequency in cycles per second (units: lb/in.).

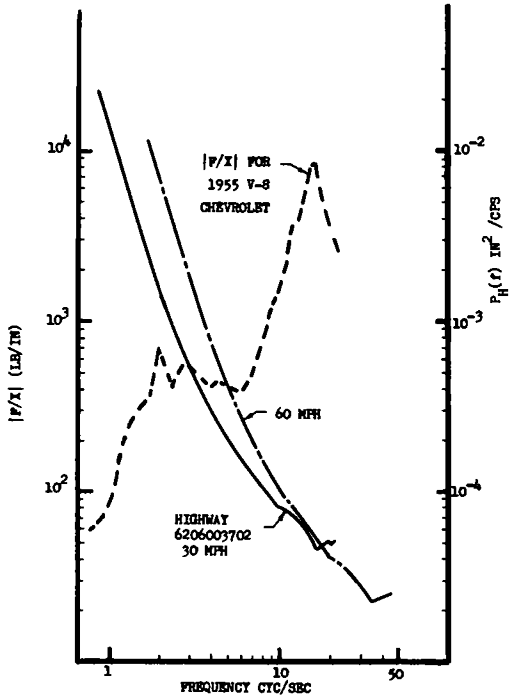


Figure 5. Pavement and vehicle characteristics shown as a function of frequency.

This procedure is shown in Figure 6. The vehicle characteristics must be squared when performing this mathematical operation. This results in extremely large peaks in the vehicle characteristic and makes the maximum values much more significant than they appear from the characteristic curve shown in Figure 1.

A typical result of these calculations is shown in Figure 7, in which the dynamic force power spectrum is plotted. The area under this curve represents the mean squared value of the force that the vehicle exerts on the highway. In addition, the curve shows the contributions to this mean squared force that are made by various ranges of frequency. From the curve in Figure 7, it is evident that two frequency bands make the largest contributions to the dynamic force. It is no coincidence that the center frequencies of these bands occur at the natural frequencies of the vehicle as determined from Figure 1. Also, a large amount of dynamic force is generated at the lower natural frequency of the vehicle as well as at the higher natural frequency. From Figure 1, it would be logical to expect the largest contribution to occur at the highest natural frequency. Allowing for the distortion that results from the use of the log-log plot shown in Figure 7, it is evident that the larger highway excitation occurring at the lower frequencies tends to develop a large amount of dynamic force at the lower frequency even though the vehicle is less responsive to this frequency. Large amounts of excitation at frequencies at which the vehicle is relatively unresponsive can produce a final result of the same order of magnitude as smaller amounts of excitation will produce at frequencies at which the vehicle is extremely responsive.

From Figure 7, it would appear that a record of dynamic force vs time would contain two predominating frequencies. This has been verified experimentally by making tire pressure measurements in which a record of tire pressure vs time was obtained for a typical passenger vehicle. This vehicle was not the same as the one shown in Figure 1 and thus exact agreement is not possible. Figure 8 shows that two frequencies predominate in the record. These two frequencies coincide fairly closely with the frequencies predicted by the power spectrum calculation shown in Figure 7. Steps are

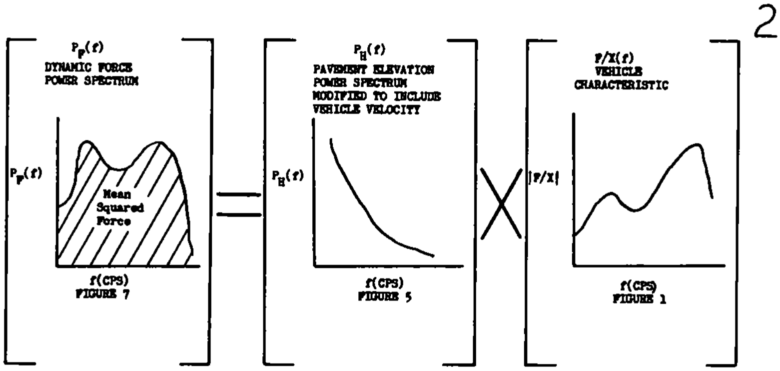


Figure 6. Procedure for computing dynamic vehicle reaction.

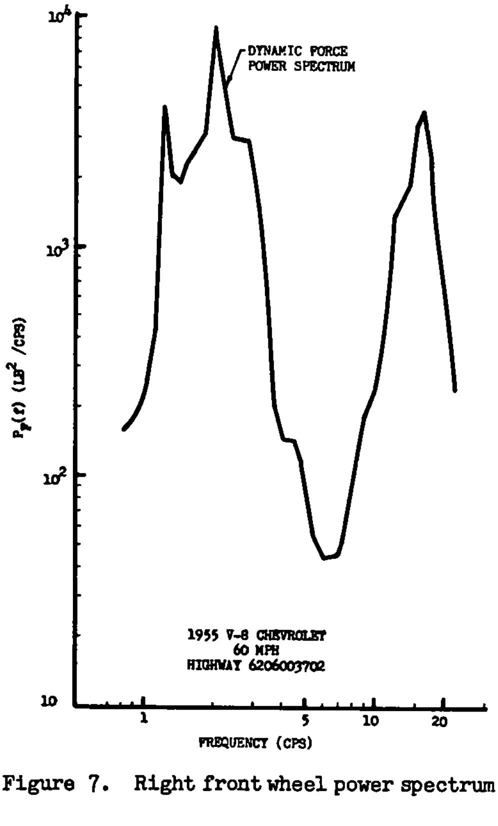


Figure 7. Right front wheel power spectrum.

being taken to obtain additional records of the type shown in Figure 8 so that an experimental check can be made of the predicted values of dynamic force obtained by using the vehicle and highway characteristics just described.

Having the mean squared value of the dynamic force it is easy to determine the root-mean squared value. Using this value, the results of operating the vehicle (shown in Fig. 1) over different highways at various speeds are shown in Figure 9. At certain vehicle velocities an increase in the speed will produce proportionally greater changes in the dynamic force than will occur at other velocities. There are also regions in which increasing the vehicle velocity will result in a decrease in the dynamic force. Using the curves shown in Figure 9, it would be possible to establish vehicle speeds that would result in the same value of dynamic force being generated for the same vehicle on different highways. Vehicle speed limits could thus be based on the allowable amount of dynamic force.

Inasmuch as root-mean squared values are shown, it is evident that larger values for the dynamic force must exist at any given velocity than those shown in the

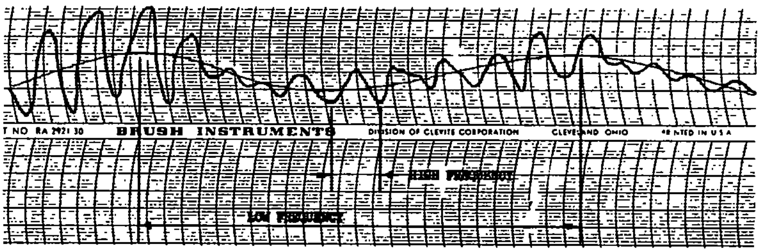


Figure 8. Record of tire pressure vs time on Highway 8086102005.

curves. It would be interesting to know how much larger these forces become and how frequently they reach these larger values. In addition, the curves are based on a pavement roughness criterion (power spectrum) that can only detect pavement variations through elevation measurements made 1-ft apart. Therefore, this calculation results in a conservative estimate of the dynamic force between the vehicle and the pavement, inasmuch as all of the pavement roughness will not be detected.

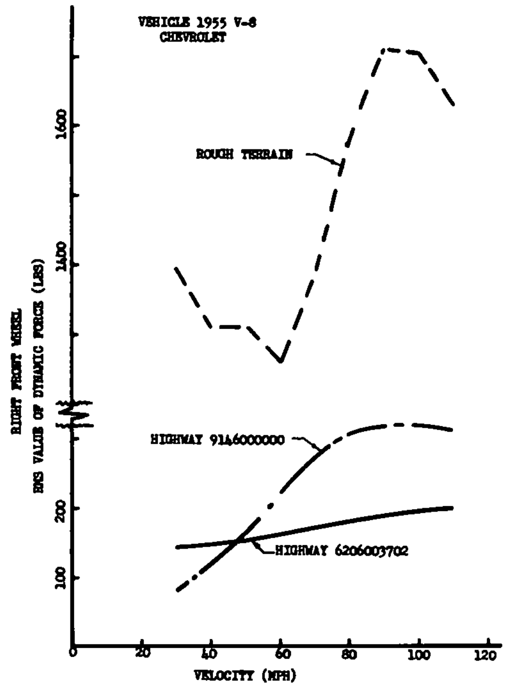


Figure 9. Root-mean squared value of dynamic force vs vehicle velocity.

## CONCLUSIONS

By using the procedures described in this paper it is possible to obtain vehicle and pavement characteristics that can be used to compute the power spectrum of the dynamic force that a vehicle will exert on the pavement at any selected velocity. Because these characteristics are determined experimentally, there are relatively few assumptions involved in obtaining the dynamic reaction. Nevertheless the question as to the accuracy of the predicted dynamic force can still be raised. Fortunately, it appears possible to check this force experimentally by using the tire pressure measuring technique mentioned in connection with Figure 9, and steps are being taken to do so.

Although the root-mean squared value of the dynamic reaction can be found, the question as to the maximum value is still unanswered. The validity of assuming that this force has a normal distribution (6) is worthy of additional consideration so that the frequency of occurrence of larger values can be estimated. This can also be studied from tire pressure measurements.

A knowledge of the dynamic reaction between a vehicle and a highway may be of practical value in establishing permissible speed limits for various classes of vehicles. Of the three quantities that influence the dynamic reaction (pavement condition, vehicle characteristic, and vehicle velocity), the velocity of the vehicle is the easiest to change.

A paramount question remains to be answered, however: what is the significance of the dynamic vehicle reaction in terms of the effect on the highway? Is a large force associated with a high vehicle velocity more detrimental than a smaller force at a lower velocity? A knowledge of the behavior of the highway under these conditions is needed.

## ACKNOWLEDGMENTS

The support of the Bureau of Public Roads through Contract CPR 11-7941 made possible the research investigation on which this paper is based. This support is gratefully acknowledged.

The authors also wish to thank the AASHO Road Test, William N. Carey, Jr., and Paul E. Irick for assistance in determining vehicle characteristics; E.A. Finney and his staff at the Michigan State Highway Department Laboratory, East Lansing, for assistance in making tire pressure measurements; the Indiana State Highway Department for obtaining elevation measurements from which highway power spectra were computed; and the Highway Research Board for including this paper in the program at the Annual Meeting.

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# Servo-Seismic Method of Measuring Road Profile

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• AT THE GENERAL MOTORS Research Laboratories a ride simulator is used for the study of ride characteristics of the many different General Motors' products. The input to this ride simulator is the road profile of actual roads whose measurements have been stored on magnetic tape. This paper is about the method that was developed for measuring this road profile.

Before starting on the discussion of the measuring, a broader understanding of the Research Laboratories' ride simulator program may be helpful. Figure 1 shows a view of the ride simulator installation. Using the road profile as an input, an analog computer produces voltages proportional to the calculated movements of the car body for any selected suspension characteristics. These voltages then are fed into electro-hydraulic valves and actuators which then produce car body movements of the desired magnitudes. With the actuators shown, the car body can be subjected to bounce, pitch, and roll. Intuitively, it can be seen that the road profile input from two tire tracks is required to obtain body roll.

With this brief background, it is apparent that accurate recording of the road profile of two tire tracks is a very important part of the ride simulator program. This paper describes the method used to measure road profile and techniques used to establish the accuracy of these measurements and attempts to correlate the frequency characteristics of the road profile signal with road roughness as experienced through a car suspension.

At the beginning of the ride simulator program, it was realized that it was necessary to measure the profile of many miles of the country's roads. The results of the usual literature search paralleled F. N. Hveem's report presented at the 39th Annual Meeting of the Highway Research Board. It soon became apparent that the methods developed by the highway people for measuring road roughness were not capable of measuring the road wave lengths required for the ride simulator program.

The methods used by the U. S. Air Force to measure runway roughness were also investigated. These included transiting and the use of a stationary projector that casts a highly collimated light beam onto an optical head moving along the runway. The optical head consisted of an array of photoelectric cells which sensed the light beam's vertical position. Both of these methods allowed the measurement of the long waves but did not have the speed and flexibility desired for continuous measurement of miles and miles of the country's roads.

It was soon concluded that a new method would have to be developed. The results of the first attempt are shown in Figure 2. This method consisted of a road following wheel held against the ground by a spring that used the towing vehicle as a reaction. An accelerometer mounted on the wheel produced an electrical signal that was integrated twice to obtain an electrical signal representing the movement of the accelerometer and thus through the wheel a representation of the road profile. However, difficulty was experienced measuring the long waves. The inadequacy of this method can be attributed to the large maximum accelerations which required the use of a 30-g accelerometer and the resulting poor accelerometer signal to noise ratio for the subtle low-frequency long-wave operation; that is, the signal representing the long-wave displacement contained so much instrument noise that the road profile signal could not be separated. Considerable time was spent trying to develop this method but it was finally abandoned.

In effect, the double integration of the accelerometer signal was the measuring of accelerometer displacement with respect to an inertial reference. Using an inertial reference appeared desirable, and this concept was carried over to the second attack

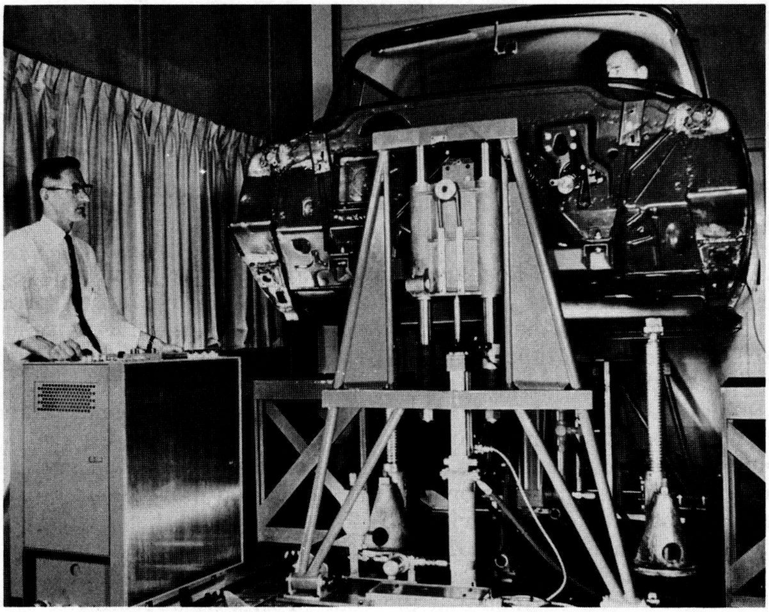

Figure 1. The General Motors vehicle ride simulator.

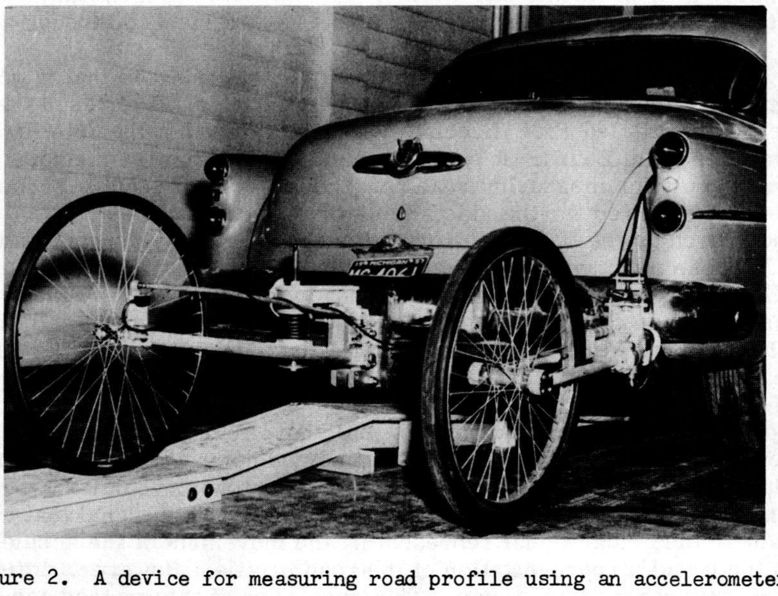

Figure 2. A device for measuring road profile using an accelerometer.

on the problem. In this approach it was decided to establish an inertial reference platform and measure the vertical displacement of the road following wheel with respect to this platform. This reference platform must, therefore, be held stationary with respect to the universe. To determine if the platform has moved, an accelerometer was placed on the platform. An acceleration signal from the accelerometer indicates the platform has moved and the double integration of the accelerometer signal shows how much. If this information can be used to move the platform physically back to its

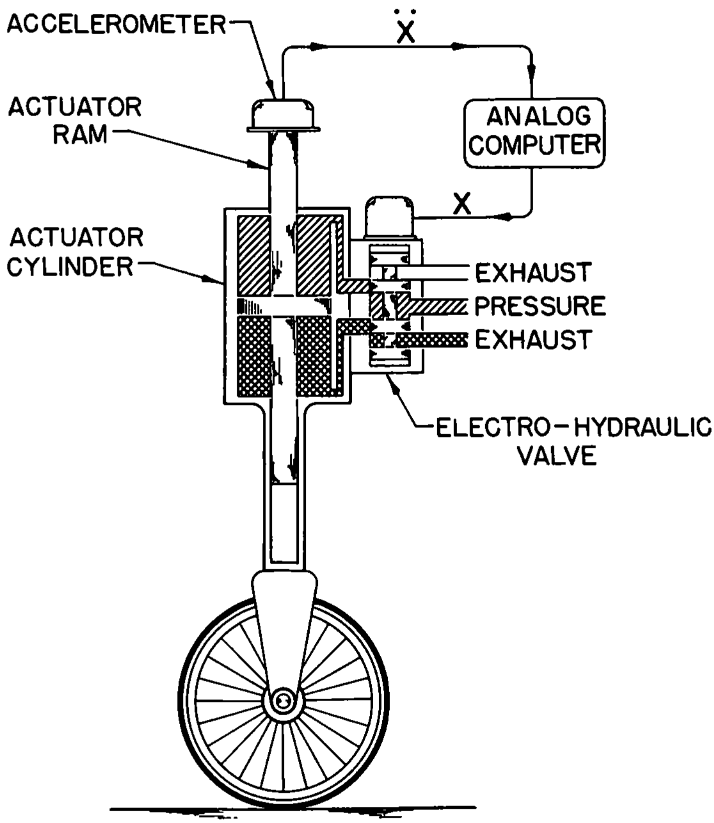


Figure 3. Schematic of Servo-Seismic road profile recording device.

original location, the inertial reference platform has been established. This platform positioning was obtained by use of an electrohydraulic valve and hydraulic actuator which produce a movement of the actuator ram that is proportional to the electric voltage supplied to the valve. Figure 3 shows how these components were combined to accomplish this. The actuator ram with the accelerometer mounted on the end is the inertial reference platform. The analog computer integrates the accelerometer signal and feeds a signal proportional to the platform displacement to the electrohydraulic valve. The valve then causes an oil flow to the appropriate chamber in the actuator. However, this is an oversimplification of the system.

Figure 4 shows a sequence with the road profile measuring wheel going over a bump. As the wheel goes up the bump, the wheel, actuator and accelerometer are accelerated upward. This accelerometer signal is integrated twice, to determine the upward displacement of these components. This displacement signal is used to cause an oil flow in the electrohydraulic valve. The signal is scaled so that the oil flow into the top chamber of the actuator and exhausted oil out of the bottom chamber causes a relative movement of the actuator cylinder and ram exactly equal to the upward movement of the actuator cylinder, thus causing the ram to remain stationary. The road profile can be obtained by measuring the displacement of the actuator cylinder with respect to the ram. The oil ports in the valve are reversed when the wheel goes down the bump. Again, this is an oversimplification of the system, but it illustrates the principle.

Figure 5 is a simplified block diagram of the control system. The circles are summing junctions either mechanical or electrical, and the blocks represent the various components. The input to the system is the actual road profile, Y, as represented by the vertical movement of the wheel hub and actuator cylinder. The first summing junction is the mechanical relationship between the actuator cylinder and the actuator ram.

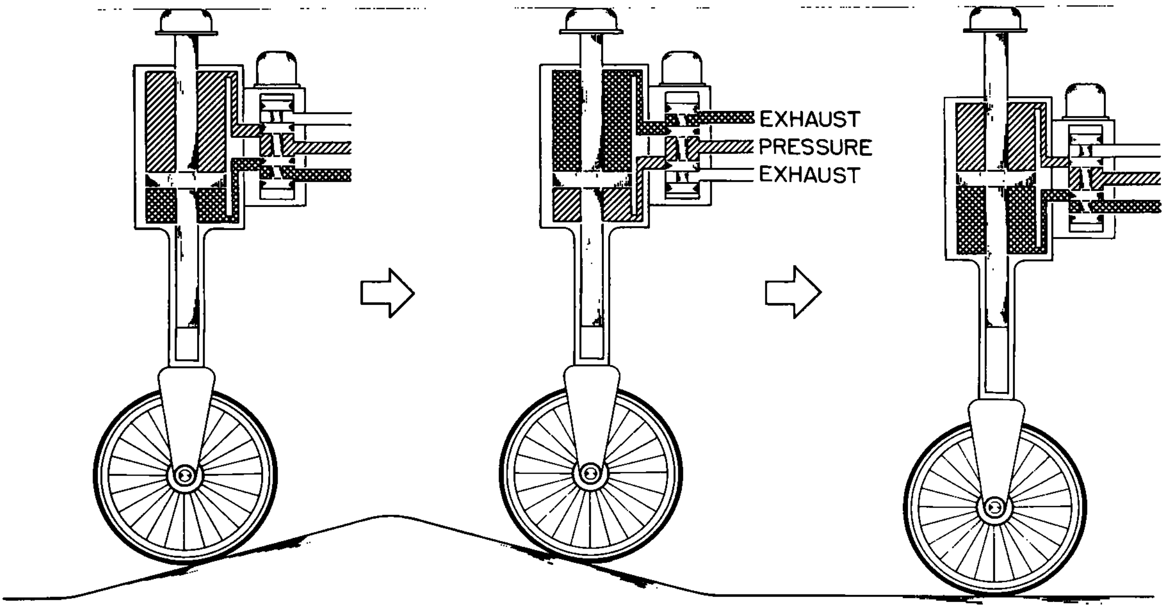


Figure 4. Sequence showing Servo-Seismic device passing over bump.

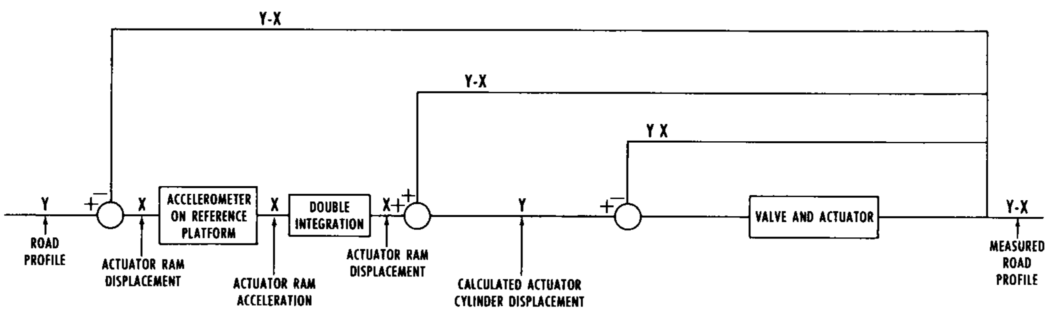


Figure 5. Simplified block diagram of Servo-Seismic system.

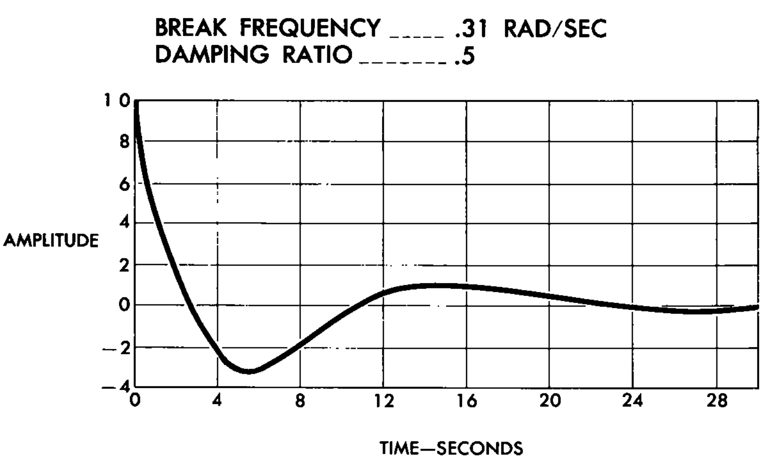


Figure 6. Transient response of third order filter to unit step input.

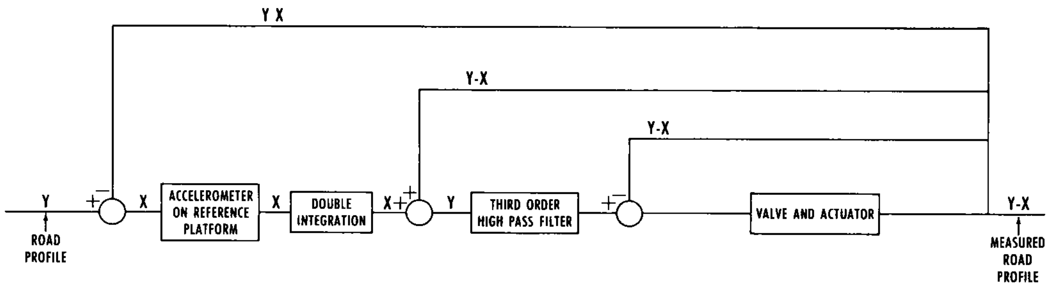


Figure 7. Block diagram of system showing addition of third order filter.

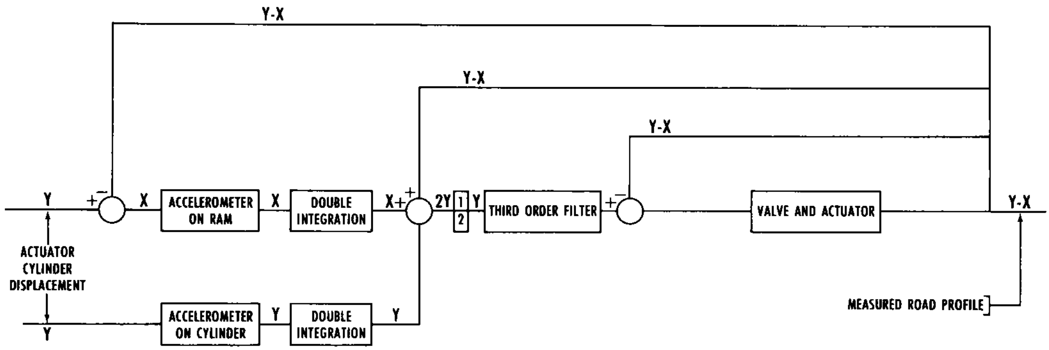


Figure 8. Block diagram of Figure 7 with parallel open loop added.

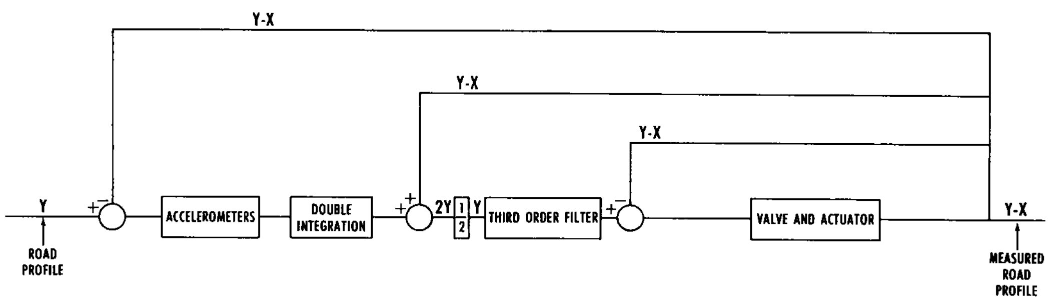


Figure 9. Simplified block diagram.

The movement of the ram, X, is the result of this summation. Because the displacement of X with respect to an inertial reference cannot be measured directly, an accelerometer is used to measure $\ddot{X}$ and a double integration performed to obtain X. The second summing junction is electrical and adds the measured absolute movement of the ram to the relative movement of the cylinder and ram, Y-X, to determine the movement of the cylinder. Using the valve and actuator as a position feedback servomechanism, an oil flow is caused in the actuator that will allow the actuator cylinder to move a displacement, Y, with respect to the actuator ram. If the measurement of X and resulting computer calculation of Y are correct, it can be seen that the reference platform returns to its original position. In fact, if the system has no time lags, the reference platform might never move.

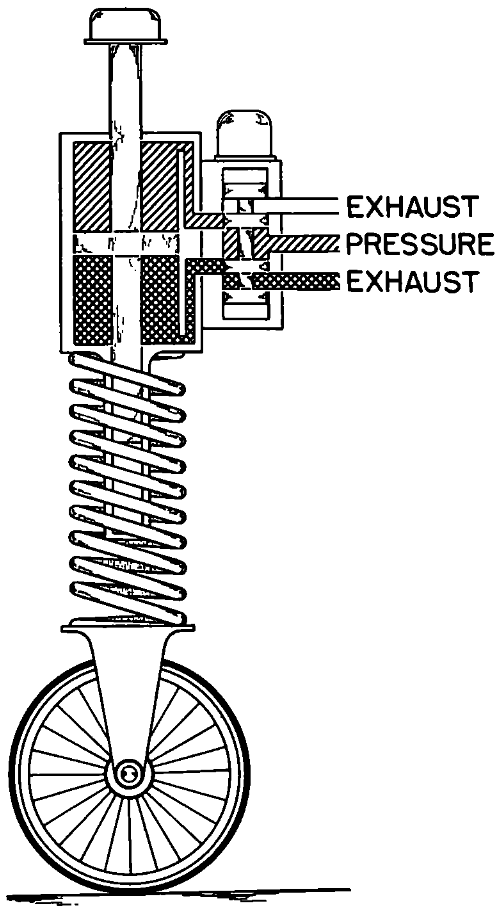


Figure 10. Servo-Seismic device incorporating spring between wheel and actuator.

It may be apparent by now that the reference platform could plow into the side of a hill or at any rate run out of actuator travel if the road amplitude were large. With the road profile trailer towed down the road at a fixed speed, it is necessary to hold the reference for a certain period of time to measure road waves of a certain length. Conversely, it is necessary to return the actuator ram to the center of the cylinder as a function of time. In effect, it is desired to filter the low frequency components or provide a "high-pass filter." Figure 6 shows the transient response of a third order filter to a step input. The 0.31 rad per sec break frequency and 0.5 damping ratio were used in some of the work presented later. Figure 7 is the same block diagram as shown before with the addition of this high-pass filter on the signal to the valve. This block diagram can be described as a closed-loop system because it strives to hold the reference platform stationary and has an error signal if it is unsuccessful. In parallel with this closed-loop system, there is also an open-loop system which converts the block diagram to a multiple input system. Figure 8 is the same block diagram as Figure 7 with the lower branch added. In this system in addition to forming Y as the result of measuring X, an accelerometer is mounted on the parts of the system which follow the road profile and obtain Y directly by integrating this signal. The signal after the second summing junction then is 2 Y and must be halved to maintain proper scaling. Various other proportions of the signal sources can be used as conditions require. This second branch or second source of Y was added to improve the low frequency characteristics of the system by allowing the measuring of an acceleration signal that was considerably larger than the accelerometer noise.

By making the assumption that the two accelerometers are identical, the block diagram can be further simplified as shown in Figure 9. By mounting an accelerometer on the actuator cylinder, large amplitude accelerations are again a problem. To reduce the acceleration level, a spring was inserted between the wheel and the actuator which, in effect, acted as a second order low pass filter. Figure 10 shows the final configuration with the road profile being measured as the relative movement of the wheel hub and actuator ram. Figure 11 shows the complete system including road profile trailer; towing truck; a 20-amplifier, transistorized analog computer; and various other components. The road profile trailer includes the road-following wheel, accelerometers, the electrohydraulic valve, hydraulic actuator, and the road-measuring potentiometers. Figure 12 shows a block diagram of the complete system with the transfer functions for the individual components shown as functions of the LaPlace operator, s. The input to the system is the road profile, now called W, and the output is the measured road profile, X-W. The quantity X-W can be obtained as the summation of two potentiometer signals Y-W and X-Y. It is obtained more directly by a potentiometer between the reference platform and wheel hub.

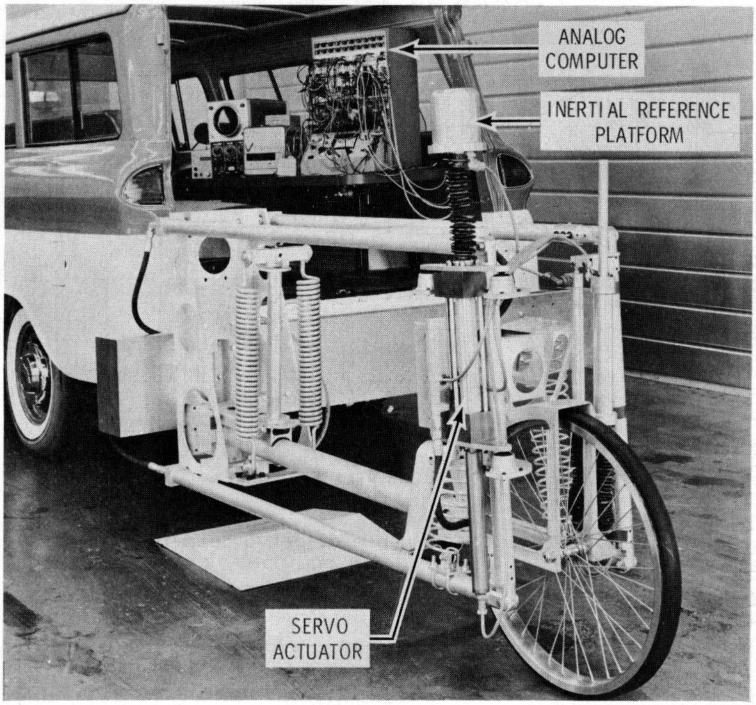


Figure 11. Photograph of complete Serve-Seismic road profiling system.

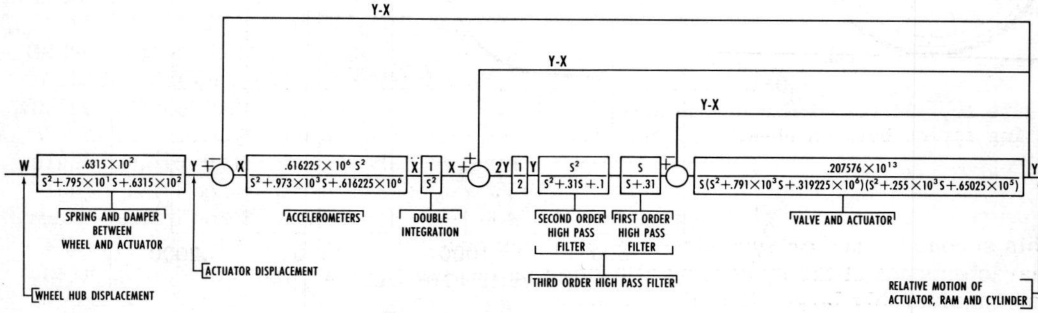


Figure 12. Block diagram showing transfer functions of system components.

A digital computer program was used to obtain the frequency response of the complete system. The frequency response in the form of a "Bode" plot is shown in Figure 13. Both amplitude and phase lag are plotted as a function of frequency. The amplitude is the ratio of the output of the system, X-W, to the input, W, expressed in decibels (db). The phase lag is the amount the output lags the input in degrees. The amplitude is 0 db or an amplitude ratio of 1 above the break frequency of 0.31 rad per sec. For the amplitude ratio to be 1, the reference platform movement, X, must approach 0. Figure 13 also shows the same system without the spring between the wheel hub and actuator cylinder for comparison. The addition of the spring produced a noticeable improvement in amplitude ratio and phase relationship at higher frequencies. At the break frequency of 0.31 rad per sec, the output leads the input by $135^0$. At 10 rad per sec and above, there is no appreciable phase difference for the system with the actuator

SYSTEM FREQUENCY RESPONSE

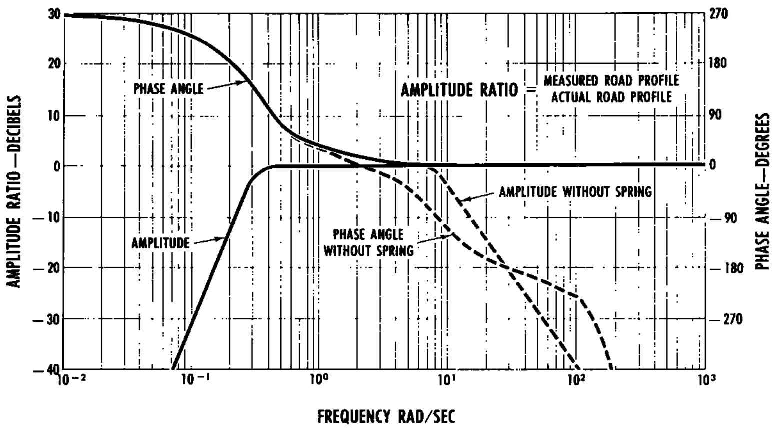


Figure 13. Frequency response of Servo-Seismic system.

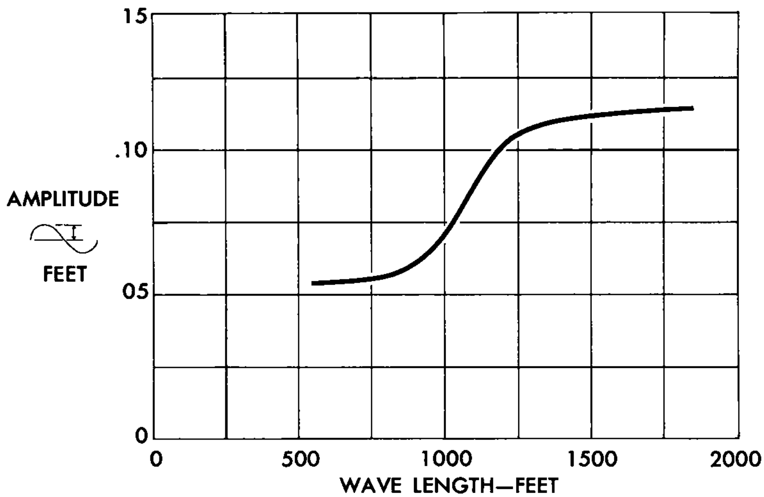


Figure 14. Sensitivity of vehicle passengers to sinusoidal road disturbances at 100 mph.

$$\frac{Y\text{-}X}{W} = \frac{38914 \times 10^{10} S^3}{[S^2 + 7\ 95\ S + 63\ 15]\ [S^2 + 981\ 8\ S + 609674]\ [(S^2 + 31\ S + 1)(S + 31)]\ [S^2 + 796\ 2\ S + 276201]\ [(S^2 + 53\ 86\ S + 26325)(S + 289\ 1)]}$$

$$\frac{Y\text{-}X}{W} = \frac{KS^3}{[\text{WHEEL SPRING}]\ [\text{ACCELEROMETER}]\ [\text{THIRD ORDER HIGH PASS FILTER}]\ [\text{SERVO VALVE}]\ [\text{HYDRAULIC ACTUATOR}]}$$

| [WHEEL SPRING] | [ACCELEROMETER] | [THIRD ORDER HIGH PASS FILTER] | [SERVO VALVE] | [HYDRAULIC ACTUATOR] |
|---|---|---|---|---|
| $\omega = 7\ 95$ RAD/SEC | $\omega = 780\ 8$ RAD/SEC | $\omega = 31$ RAD/SEC | $\omega = 525\ 6$ RAD/SEC | $\omega = 162\ 3$ RAD/SEC |
| $\zeta = 50$ | $\zeta = 63$ | $\zeta = 50$ | $\zeta = 76$ | $\zeta = 17$ |
| | | | | $\omega = 289\ 1$ RAD/SEC |

Figure 15. System transfer function.

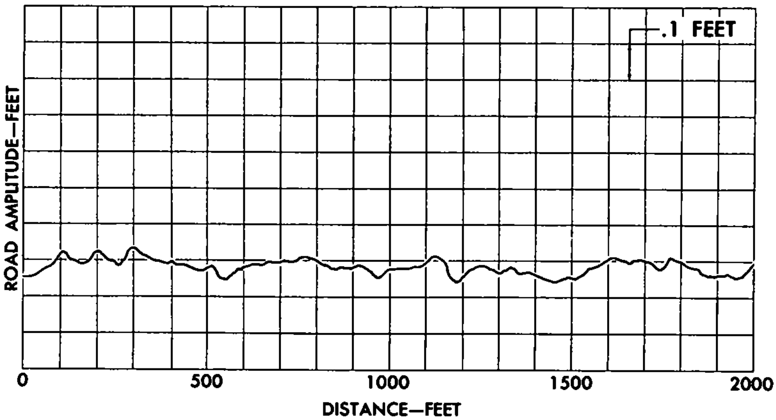


Figure 16. North-South Straightaway, G.M. Milford Proving Ground, Servo-Seismic method.

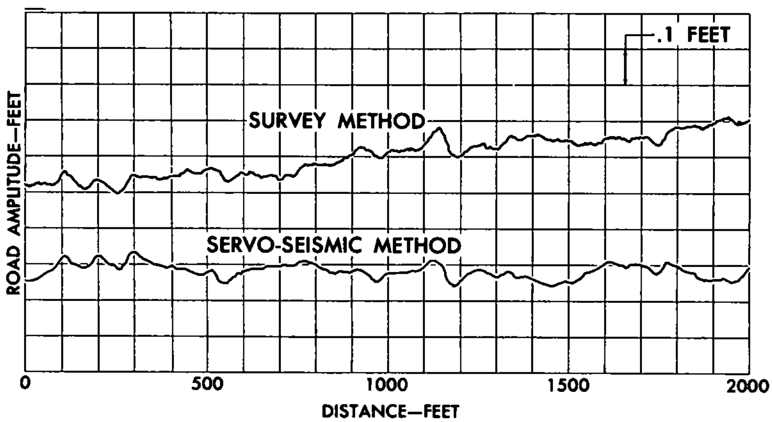


Figure 17. North-South Straightaway, G.M. Milford Proving Ground.

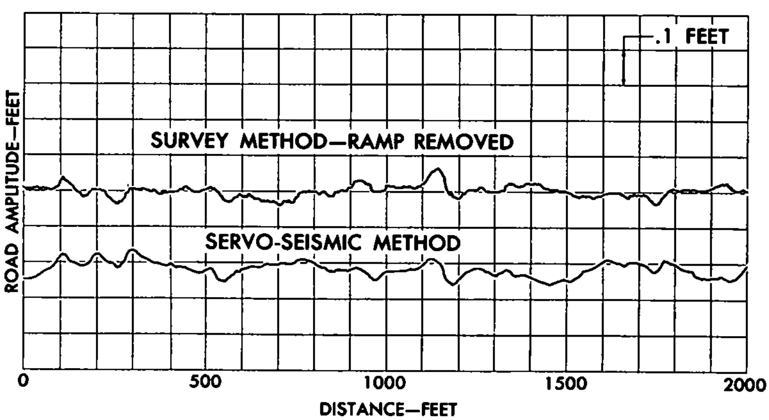


Figure 18. North-South Straightaway, G.M. Proving Ground.

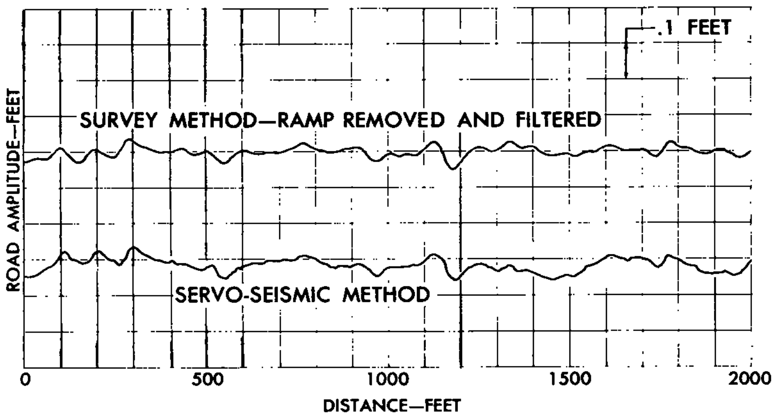


Figure 19. North-South Straightaway, G.M. Proving Ground.

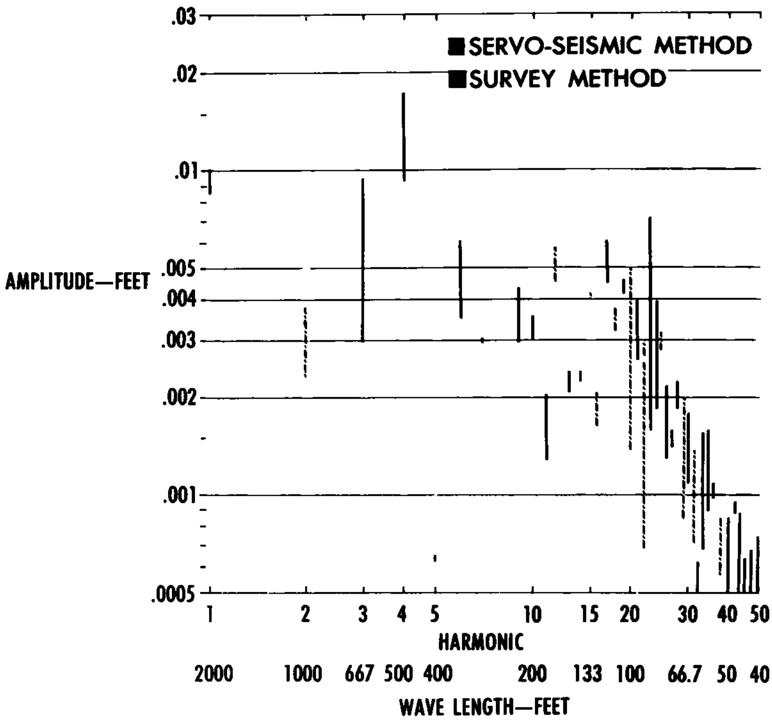


Figure 20. Harmonic analysis, 2,000-ft sample, North-South Straightaway. On this and figures following that compare harmonic analyses of two roads, the ends of each bar mark the amplitudes of the corresponding wave-length components of the two roads. Length of bar represents difference in amplitude. Shading indicates which road contains the larger amplitude. For example, at the 500-ft wave length, the "surveyed" road amplitude is 0.009 ft and the Servo-Seismic, 0.018 ft.

sprung. The effect of the low-frequency phase difference on the accuracy of the system was considered but was found to be insignificant. Figure 13 shows that the selection of the 0.31-rad per sec break frequency for the third order filter was one of the more important decisions.

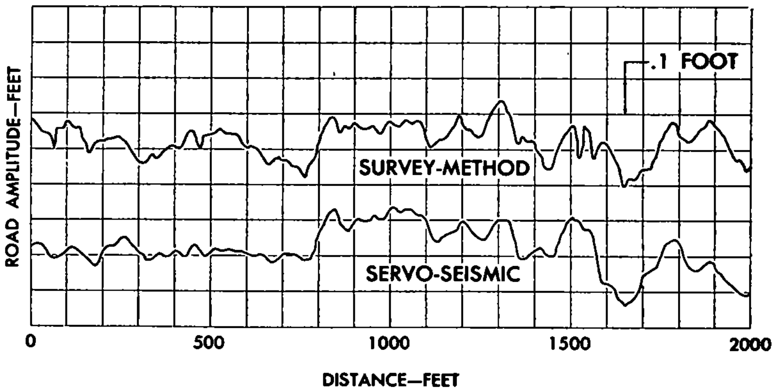


Figure 21. Military Straightaway, G.M. Milford Proving Ground.

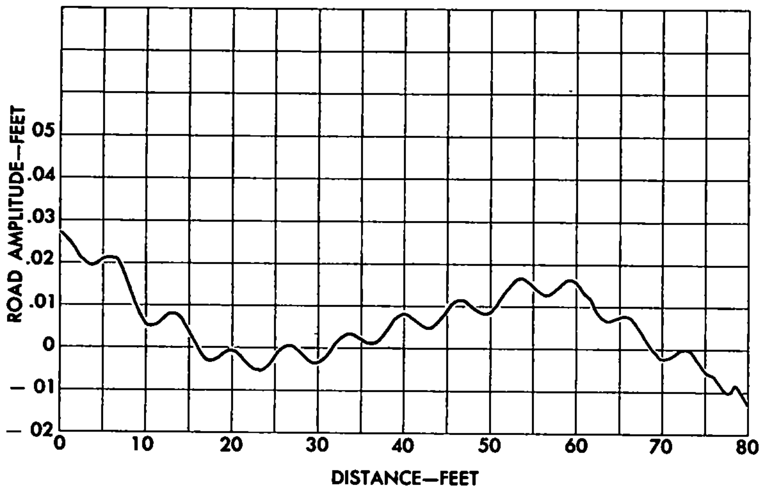


Figure 22. North-South Straightaway, G.M. Milford Proving Ground, with out-of-round wheel.

Some of the considerations that enter into the decision are maximum road wave length to be measured, the amount of travel available in the ram, the ability to hold the wheel on the road, and the ability of the human body to sense change in position as a function of time. In effect, the reference platform is held stationary for a certain period of time after which the ram returns to the center of the cylinder. As a result, the length of wave measured with respect to this reference platform is a function of the filter time constant and the velocity at which the wheel and reference platform are moved down the road. Now the problem of actuator ram travel is apparent. If the wave amplitude exceeds the actuator travel, the trailer velocity must be reduced or the filter break frequency increased. An actuator travel of ± 5 in., which is the maximum commercially available, is used on the trailer shown in Figure 11. The ability to hold the wheel on the ground and the wheel strength appear to be the limits on top trailer recording velocity. A wheel hold-down force of ten times the weight of the wheel assembly is currently being used. No difficulty is experienced in measuring most roads at 20 mph. The fourth consideration is the human body's ability to sense change in position as a function of time. This was determined subjectively using the ride simulator.

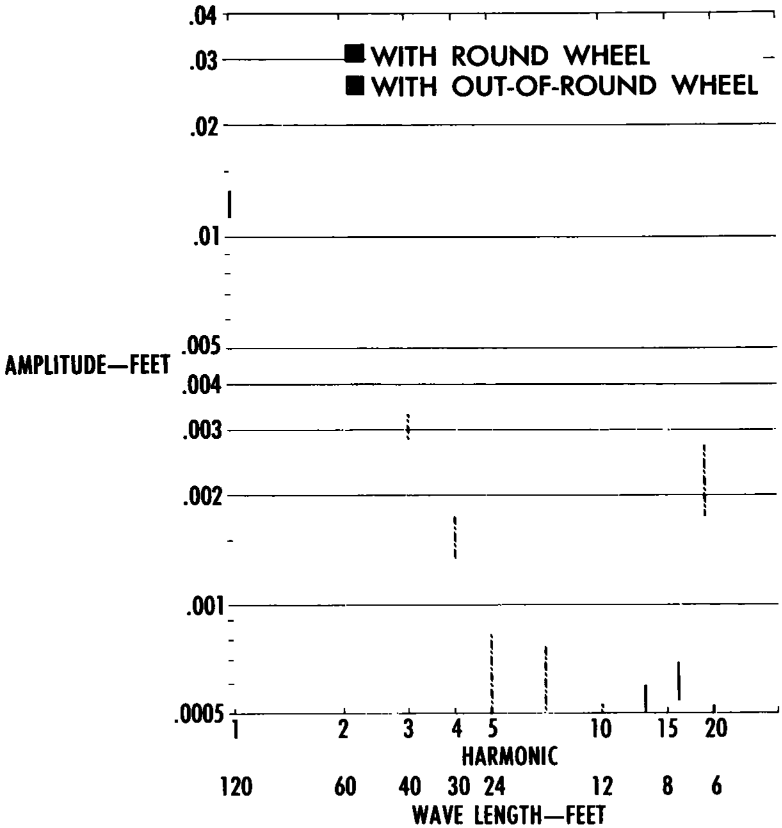


Figure 23. Harmonic analysis, 120-ft sample of North-South Straightaway. (See caption, Fig. 20).

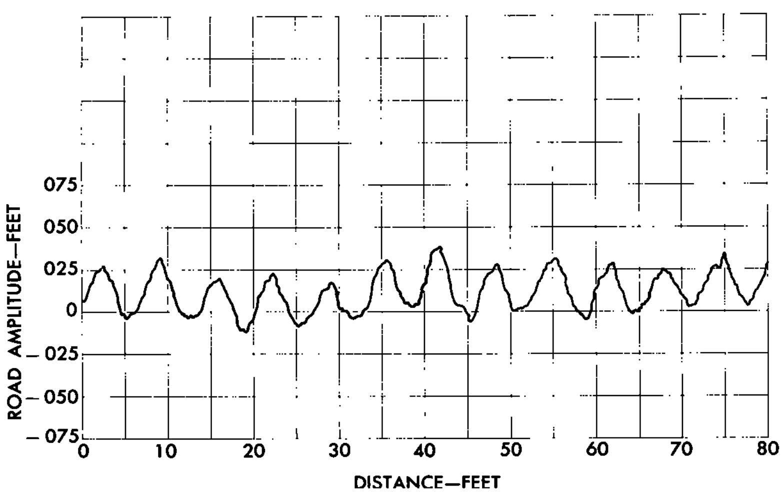


Figure 24. Sine Road, G.M. Milford Proving Ground; wave length—6 ft, wave amplitude—0.03 ft peak to peak.

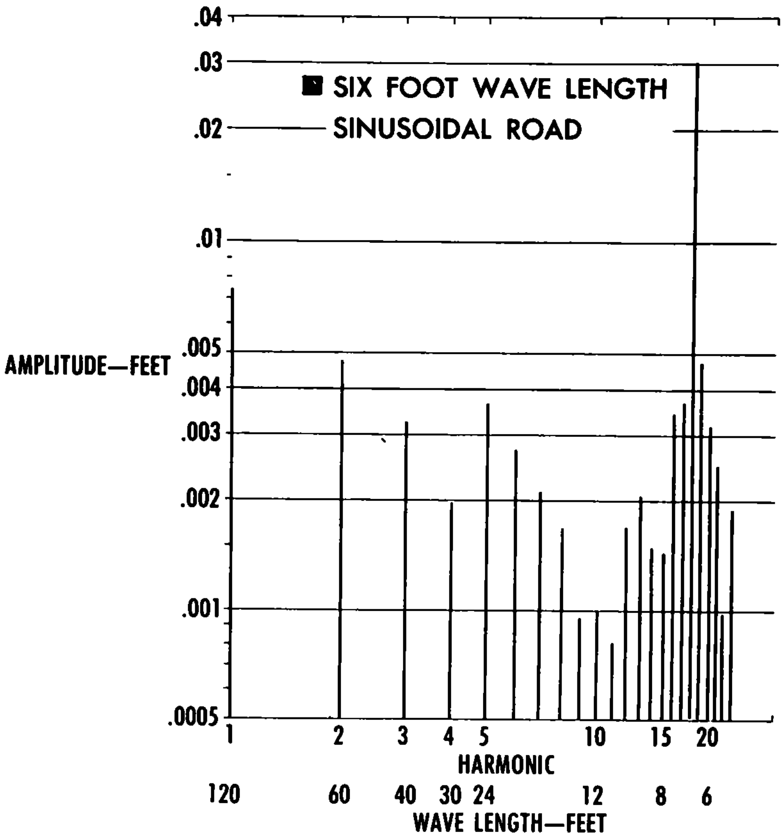


Figure 25. Harmonic analysis, 120-ft sample.

A passenger in the car body of the ride simulator was subjected to an input of sine waves to determine what wave amplitude could be perceived at an equivalent car speed of 100 mph. The test was run by holding wave frequency constant and increasing wave amplitude until the subject could tell he was moving. Superimposed on these low-frequency waves was a low-amplitude white noise which was held constant. Figure 14 shows the preliminary results of this study. This curve indicates that at car velocities of 100 mph, a 500-ft wave with an amplitude in excess of 0.05 ft is of importance in vehicle-ride studies. The subject becomes less conscious of wave amplitude as the wave length increases but is still aware of a wave amplitude in excess of 0.12 ft in 1,800-ft waves.

The requirement to measure wave lengths of this magnitude is certainly severe but it may be possible on roads designed for 100-mph traffic where wave amplitude is low. However, for the initial work, it appears that a recording velocity of 20 mph, break frequency of 0.62 and a damping ratio of 0.5 will allow the measurement of most roads with the actuator travel available. This will allow the measurement of wave lengths up to 300 ft which at 100-mph car velocity produce input frequencies to the car below the natural frequencies of all current road vehicle suspensions.

Now that this servo-mechanism network has been developed to measure road profiles, its stability must be considered. The transfer function for the entire closed loop is shown in Figure 15 with the denominator or characteristic equation factored. Because the characteristic equation has no positive roots, the system is stable. The damping ratio of the roots corresponding to the hydraulic actuator appears rather light, but has caused no stability problem. If necessary, actuator damping can be increased by

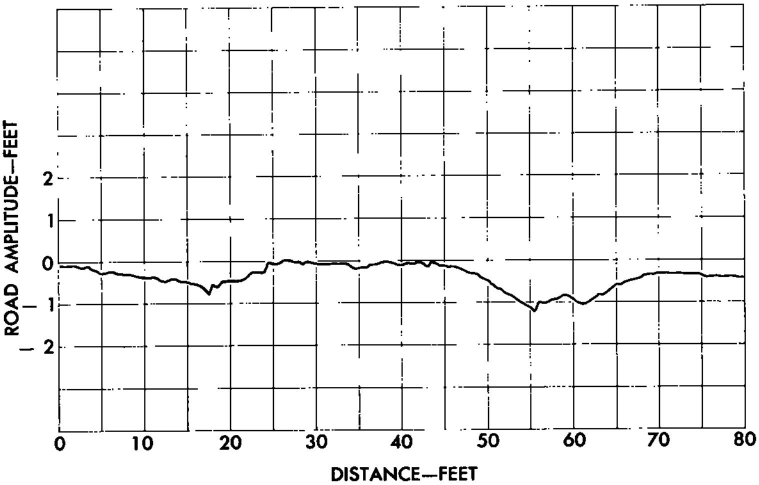


Figure 26. Military Straightaway, G.M. Milford Proving Ground.

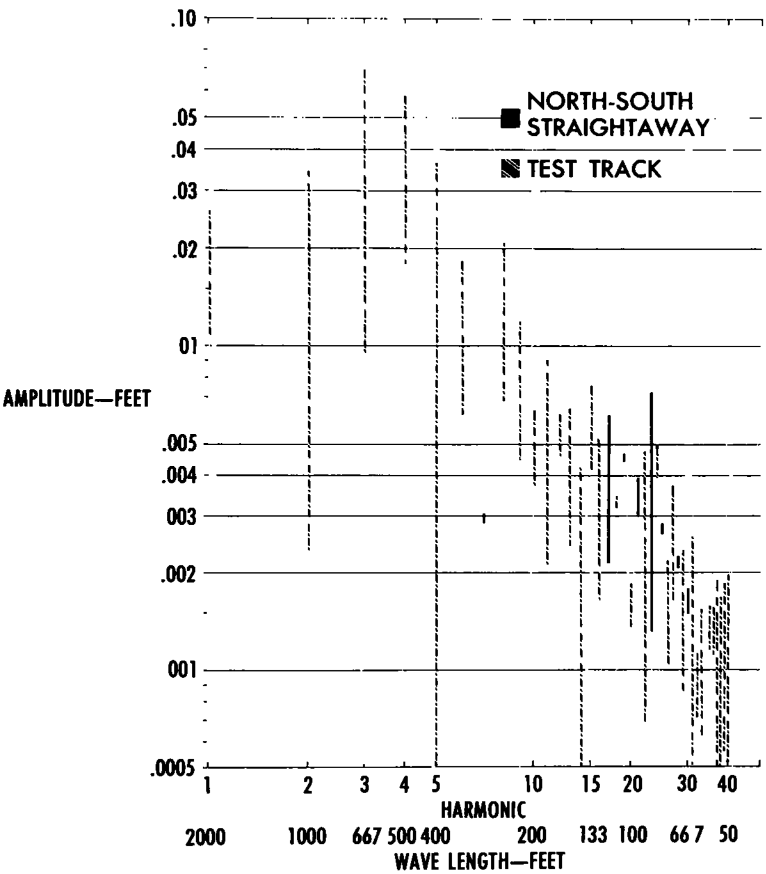


Figure 27. Harmonic analysis, 2,000-ft sample. (See caption, Fig. 20.)

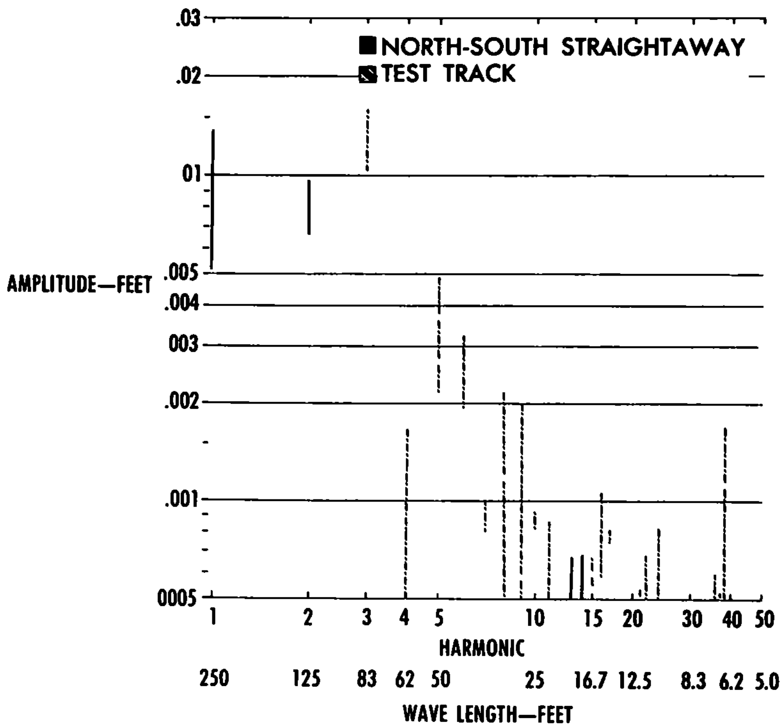


Figure 28. Harmonic analysis, 250-ft sample. (See caption, Fig. 20.)

reducing the valve gain with little effect on the over-all system. The actual stability analysis was more complete than this and included root locus plots from a digital computer program.

In the over-all evaluation of the profiling device to determine its ability to capture true road profiles, the philosophy was adopted that no better test could be made than to compare the record made by the device directly with a profile of the same road as obtained by conventional surveying methods. This procedure would take into account extraneous effects that could otherwise be overlooked, such as the pitching and bouncing of the towing vehicle, electronic disturbances, bouncing of the contour-following wheel, and tilting and vibrating of the accelerometers. A satisfactory result over a large enough sample of roads would be a sufficient demonstration of the adequacy of the system.

Some difficulties existed, such as ensuring that the recorded path was the same as that surveyed, and accounting for the intentional filtering by the Servo-Seismic method. Variations in the profiles produced by these factors should not be attributed to system error.

Figures 16 through 19 show the results of a comparison test, in this case, a 2,000-ft stretch of the very smooth "North-South Straightaway" at the General Motors Proving Ground in Milford, Mich. Figure 16 shows the profile as recorded by the Servo-Seismic method. In Figure 17, the profile of the same stretch exactly as surveyed has been added to the first plot. Elevations were taken at 5-ft intervals. It is apparent that an over-all slope exists in the real road that is not picked up by the Servo-Seismic device. This was to be anticipated and Figure 18 shows a comparison of the two profiles after removing this "ramp" characteristic from the surveyed profile. In addition, wave components less than 40 ft in length have been eliminated. This facilitates examination of the longer wave components that were of concern in this instance. In Figure 19, the surveyed profile has been further subjected to a computer approximation of the third

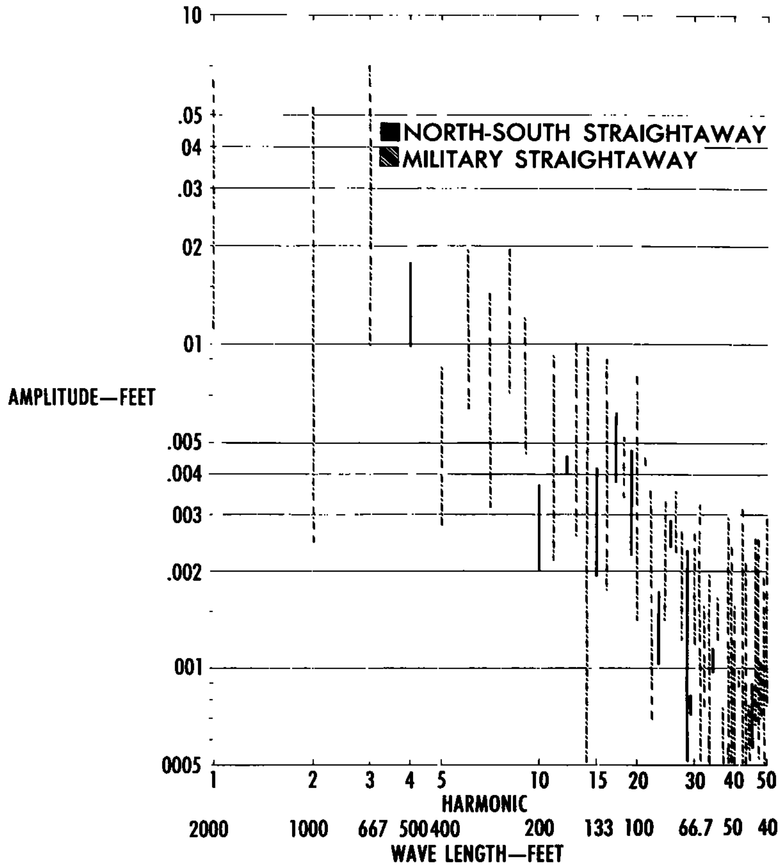


Figure 29. Harmonic analysis, 2,000-ft sample. (See caption, Fig. 20.)

order filter of the Servo-Seismic system. This has the effect of diminishing the amplitude of road components longer than approximately 600 ft.

The agreement appears to be fairly good. The sensitivity of the Servo-Seismic to long waves of small amplitude has been demonstrated here. In this range, the response is due primarily to the servo-portion of the system. Harmonic analyses of the two 2,000-ft lengths are shown in Figure 20 and also compare favorably.

Figure 21 compares the recorded and surveyed profiles of a considerably rougher stretch of road than the North-South Straightaway, 2,000 ft of the Military Straightaway at the Proving Ground. Amplitudes of the irregularities range up to three times as great as in the previous, smooth road. Here again the agreement is seen to be fairly good. In this run, the vehicle and trailer were subjected to more roughness than in the first, providing an indication of the magnitude of these effects. However, on the rougher road, small variations in path followed can result in considerable differences in profiles. Differences of this type cannot be distinguished from errors produced by deficiencies of the profiling systems.

To obtain a surveyed profile that would include a reasonable representation of the sharper irregularities of the order of 2 to 20 ft in wave length, it would have been necessary to take readings at intervals of approximately $\frac{1}{10}$ of the shortest wave length, which for 2-ft waves would be 0.2 ft. Furthermore, amplitudes in this range tend to be smaller than in the longer wave lengths and more resolution is required than is readily available in conventional surveying procedure. Instead, to verify the performance of the Servo-Seismic system for these shorter wave lengths, a recording wheel

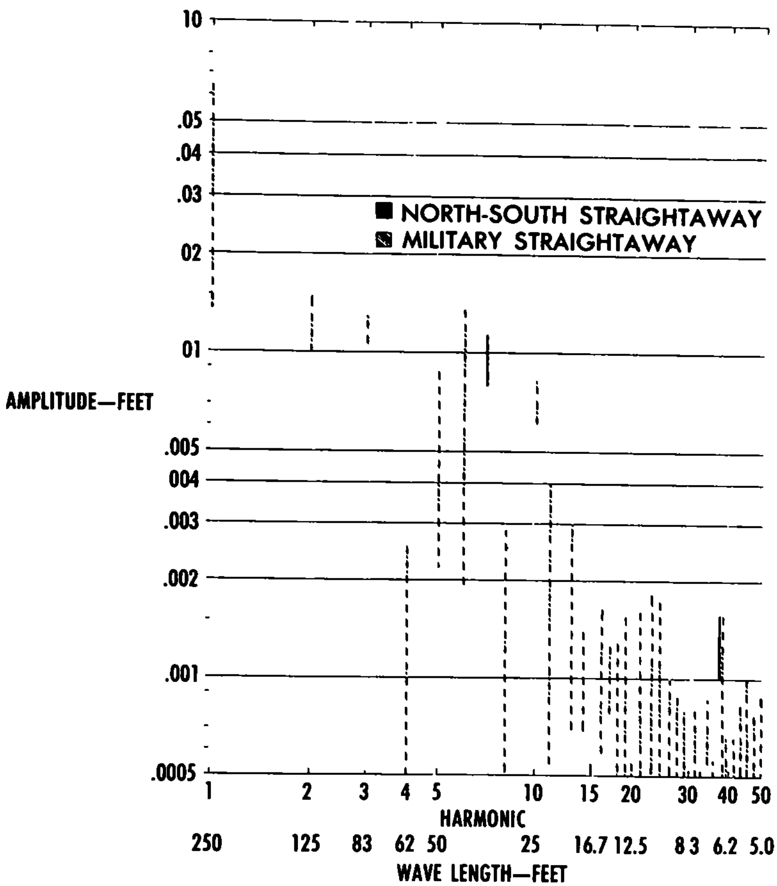


Figure 30. Harmonic analysis, 250-ft sample. (See caption, Fig. 20.)

that was made deliberately out of round was installed. The resulting effect, a repeating wave approximately 6 ft long, equal to the wheel circumference, is clearly shown in Figure 22, a 120-ft section of the North-South Straightaway. A spike in amplitude also appears at the 6-ft wave point in the harmonic analysis of this section (Fig. 23). The installed out-of-roundness of the wheel totaled 0.120 in. but was modified during the run by the compression of the rubber tire. Lesser out-of-round effects can be noted even in runs with supposedly round wheels.

A second check of system performance for the shorter wave lengths was made by attempting to record the profile of the Proving Ground "Sine Road." The profile of this road consists of continuous sine waves 6 ft long and approximately ½ in. high. These waves are faithfully reproduced in the recorded profile, 120 ft of which is shown in Figure 24. Their presence is also clearly reflected in the harmonic analysis of this section (Fig. 25). An 80-ft section of the Military Straightaway to expanded scale in Figure 26 shows typical shorter wave length irregularities.

Figures 27 through 32 compare various types of road surfaces with the North-South Straightaway as a standard. In each case, harmonic analyses of 2,000- and 250-ft stretches are presented. Characteristic differences can be discerned. The general slope appears to be about the same on all plots and suggests that the average amplitude of the wave components falls off as the square of the wave length. This implies that the acceleration amplitude introduced by these waves as input to a traversing vehicle is constant for a particular road for all wave lengths. This has also been suggested in previous investigations.

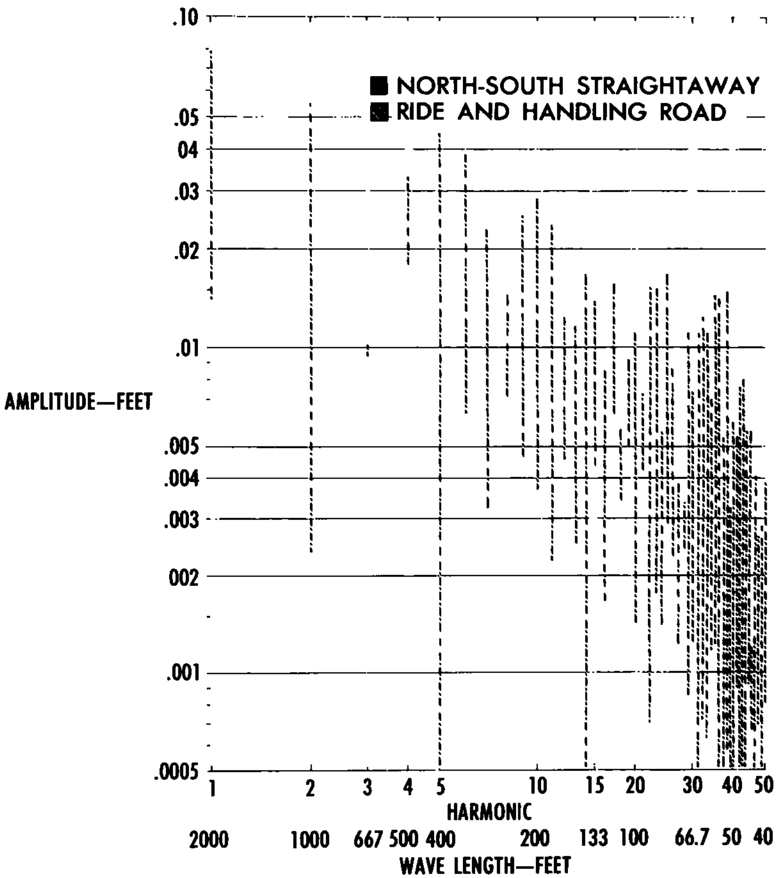


Figure 31. Harmonic analysis, 2,000-ft sample. (See caption, Fig. 20.)

In general, amplitudes contained in the test track plot (Figs. 27 and 28) appear to be somewhat larger than those in the Military Straightaway (Figs. 29 and 30). Ride and Handling Road amplitudes (Figs. 31 and 32) are clearly greater than either. Of course, the North-South Straightaway amplitudes are the smallest of all. This order conforms to the general impression of relative riding quality gained by driving an ordinary passenger car over these roads. Every plot shows a small spike of amplitude at the 6-ft wave length point. This can probably be attributed to inaccuracies in the contour-following wheel as previously mentioned, but it is also possible that actual components present in the roads may be partially responsible. Refined techniques of analysis may isolate these and other differences more distinctly.

The authors feel that the Servo-Seismic system offers a quick, reasonably accurate method of obtaining road profile data in a form convenient to rapid computer analysis. Future plans include study to develop the correlation between the recorded profile and various properties of the road itself, particularly its "riding" qualities. Hopefully, statistical, frequency, or similar methods of analysis will yield an easily interpreted mathematical criterion by which roads may be quickly and objectively evaluated.

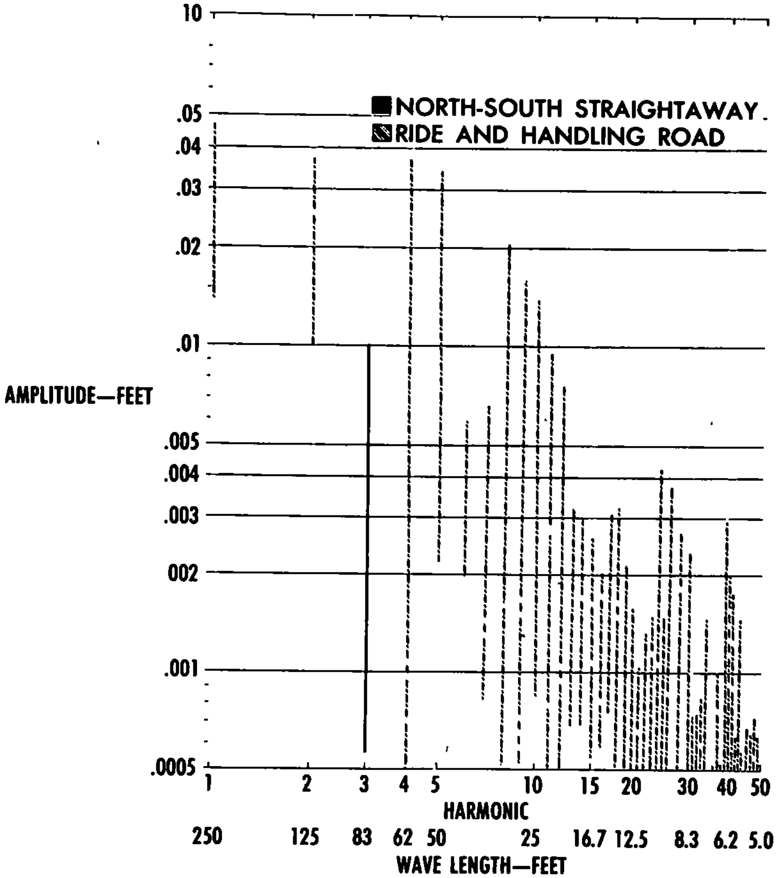


Figure 32. Harmonic analysis, 250-ft sample. (See caption, Fig. 20.)

# Experience with a BPR-Type Roadometer In Illinois

W. E. CHASTAIN, Sr., and JOHN E. BURKE, respectively, Engineer of Physical Research and Assistant Engineer of Physical Research, Illinois Division of Highways

The Illinois Division of Highways in 1957 constructed a road roughness indicator ("roadometer") patterned after the device introduced by the Bureau of Public Roads in 1941. Numerous modifications were made in adapting the device for use in Illinois. After an extensive series of tests, the Illinois instrument was placed in regular service recording the smoothness of new and old pavements beginning in 1959. The recently developed use of road roughness indicators in furnishing measurements that assist in estimating the present serviceability of pavement under the concept originating at the AASHO Road Test has greatly enhanced the value of these devices. This paper describes the various modifications made by Illinois in constructing its "roadometer," tests to which it has been subjected, and its use in rating Illinois pavements under the present serviceability concept following correlation of the device with the AASHO Road Test profilometer.

• IT IS ONLY natural that the emphasis the traveling public places on the riding quality of pavements has caused the highway engineer to search diligently for a means for making objective measurements and evaluations of this riding quality. The search has led to the development of a considerable number of measuring devices, all directed at the same objective of determining the surface smoothness of pavements, but differing widely in principle and detail.

The prototype of what has come to be the most widely used device today was developed in 1940 by the Bureau of Public Roads. Various names have been applied to this piece of equipment including road roughness indicator, roughometer, and roadometer.

The BPR-type device is basically a single-wheel trailer that is towed along the highway. The wheel is linked to the trailer by two single-leaf springs. The frame of the device is constructed of standard steel channels and has a rectangular shape. The wheel is mounted centrally in the frame, and the frame is weighted so that, if it were suspended from the towing vehicle hitch as a pendulum, the center of percussion would be in the plane of the axle. Two damping units help control the movement of the wheel with respect to the frame.

All of the foregoing construction is subject to standardization so that measurements made by various agencies operating the devices can be correlated.

As the device is towed along the pavement, irregularities in the pavement surface cause a differential vertical movement of the wheel with respect to the frame. This movement is transmitted by a wire cable to a double-acting ball-clutch integrator which converts the upward vertical motion to unidirectional rotary motion. This rotary motion is what is recorded to determine riding quality. Accuracy of the integrator is all important in the successful operation of the roadometer.

The roadometer operates on the fundamental principle that vertical oscillations of the wheel with reference to the chassis are indicative of the riding quality of a pavement. The device has proven to be reasonably sturdy and not difficult to use. Experience with reproducibility characteristics has been varied, but on the whole reasonably good. An important feature of the device is the speed at which it is operated; 20 mph, as com-

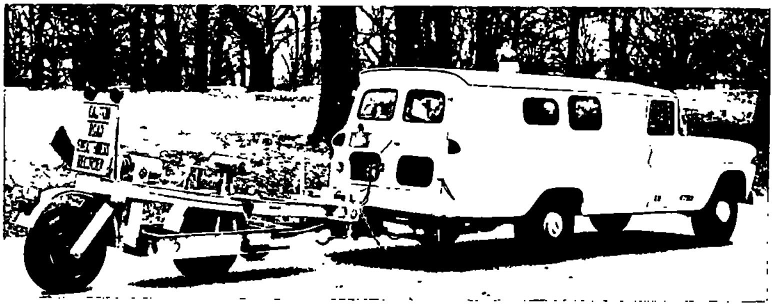

Figure 1. Over-all rear view of Illinois Division of Highways roadometer, outrigger trailer and van.

pared with the 3 to 5 mph of many other devices. This is believed advantageous for safety when operation in the regular traffic stream becomes necessary.

The Illinois Division of Highways operates two roadometers. Both have the basic features recorded in plans furnished by the Bureau of Public Roads. These features are those which were reported by J. A. Buchanan and A. L. Catudel in 1941. Although the basic details have been retained, numerous modifications have been made in both devices.

The first device placed in service by Illinois was constructed by the Division's personnel; the second, only recently acquired, was purchased already constructed. The following comments on construction and calibration details apply only to the device built by Illinois.

## ILLINOIS MODIFICATIONS OF ROADOMETER

The device constructed by Illinois was built in 1956 and first placed in service in 1957. In constructing the Illinois device, advantage was taken of certain modifications developed by the highway departments in California, Minnesota, and Missouri. Among these were the addition of profile and cumulative potentiometers to the integration system so that the roughness profile and also the accumulated roughness could be recorded on an oscillograph tape. A two-channel oscillograph as proposed by California is used in making the recordings. A multi-tapped, low-voltage, high-current, dc-power supply having the vehicle battery as its source is used to operate the oscillograph in place of dry cells that have been used by others.

Printing counters are used on the Illinois device to record counts under conditions where the details furnished by the oscillograph records are not needed.

Stepping switches activated by the revolving roadometer wheel are used to actuate automatically the recording equipment on the Illinois device to record inches of roughness at the end of each mile of travel on long runs. These switches cause the oscillograph pens to reverse at the end of each mile of travel, with each of the two recording pens serving in alternate miles as cumulative recording pens and profile recording pens. The switches also cause the recording counters to print out the measured number of inches of roughness as each mile of travel is completed.

Minor modifications include the use of open-type ball bearings with grease cap and shield to replace the double-shielded bearings, and the use of universal joints instead of ball socket joints on the damper units as shown on the Bureau of Public Roads plans.

A major innovation developed by the Illinois Division of Highways is a single-wheel outrigger carrier trailer which eliminates the necessity for carrying the device in the towing vehicle when not in use.

The roadometer travels within the outrigger trailer, and is connected to the trailer by the standardized roadometer trailer hitch. The outrigger trailer is connected to the towing vehicle by a special hitch in which a transverse lead screw extending across a specially designed bumper on the vehicle allows the device to be moved to either wheelpath of the traffic lane by the turn of a crank.


Figure 2. Close-up view of Illinois Division of Highways roadometer and outrigger trailer and van.

The outrigger trailer has an over-all width of about 33 in. and does not interfere with other traffic while the device is recording in the wheelpath positions. A cable hitch is used to raise the roadometer free of the pavement for travel when not in use and for storage. Raising or lowering of the device can be done in a few seconds.

Figures 1, 2 and 3 show the Illinois Roadometer.

## ACCURACY OF RESULTS

The Illinois device, from the beginning, has shown good reproducibility of results. Static calibrations, and field calibrations over measured lengths of pavement, have shown little deviation that could be attributed to changes in the recording characteristics of the equipment.

Static calibrations of the Illinois device made by the usually accepted procedures on an average of about once every six weeks while the roadometer is in service have shown no important changes in response of the device. Deviations from theoretical values have usually been less than 2 percent and in only one instance reached 10 percent.

The Illinois device is calibrated dynamically on a 1-mi section of rigid pavement constructed over 25 years ago. This pavement once served a high volume of traffic, but has been used only as a local road serving a few vehicles per day during the period in which it has been used for calibration purposes. During the four years of calibrating the Illinois device on an average of about once a month, roughness index values have been found to range between 106 and 116 in. per mi except in a few isolated instances. No permanent increase or decrease in readings during the four-year calibration period has been discernible.

Reading has also been repeated at less frequent intervals on other pavements of varying roughness, from very smooth to very rough, without important changes in the recorded values.

The reliability of the Illinois device is believed to be due to several factors. One of the most important is believed to be the protection that is given the equipment when not in actual operation. The outrigger carrier trailer system has been designed in such a way that the roadometer can be raised easily from the pavement and supported by the trailer when not in use. There is little reason for a careless operator to tow the recording device over rough pavements at high speeds, causing excessive wear and even damage to the relatively delicate mechanism.

Keeping the device suspended during periods of storage also removes the tendency toward the forming of flat spots on the tire. Several agencies have noted that flat spots will cause erratic readings.

Figure 3. Interior view showing instrumentation in Illinois Division of Highways roadometer van.

A frequent source of trouble recognized by users of the BPR-type roadometer is the mechanical integrator. Extreme care was used in the manufacture of the Illinois integrator, with all parts machined to the closest reasonable tolerances. Careful attention is given to ordinary maintenance of the integrator, and each winter it is completely disassembled, cleaned, and oiled. The behavior of the integrator is believed to be responsible in large measure for accuracy of results.

The entire roadometer is also cleaned and oiled each winter, the wiring is checked, and parts that show an indication of serious wear are replaced.

Breakdowns of the equipment have not been uncommon. These, however, do not cause concern as to the reliability of the results. The malfunctioning of the equipment is evident and false readings that would lead to erroneous analyses are not introduced. Components that have proved most troublesome have been the wheel revolution counters, the printing counters, and several of the relays in the system.

## PRESENT SERVICEABILITY INDEX

The roadometer furnishes a record of the cumulative upward movement of a spring-mounted wheel and axle with respect to a frame, over a measured distance of travel. The vertical movement is measured in inches and the distance in miles. The resultant inches of vertical movement per mile of travel is termed the "roughness index." The roughness index, which is simply the inches of differential movement of a spring-mounted wheel with respect to its carrying frame, per mile of travel of the wheel, is an abstract number. In itself it tells nothing about the riding quality of a pavement. To be of value the roughness index must be related to highway user opinion of riding quality.

The problem of establishing a working relationship between the results of roadometer measurements and highway user opinion of pavement riding quality has been a difficult one, and a major reason for slow acceptance of the device. Fortunately, the need for developing a system of rating pavements of the AASHO Road Test led to what appears to be a significant step forward in the solution of this problem. This may prove to be one of the important by-products of the road test.

Under a pavement serviceability-performance concept developed on the AASHO Road Test project, a system was devised for rating any pavement's ability to serve the traveling public on a scale ranging from 0 to 5. The serviceability, as it is called, relates only to the time of rating. It is the average rating that would be applied to a pavement by highway users. It is the answer to the question of how well on this scale of 0 to 5 this pavement is able to serve high-speed, high-volume mixed passenger and truck traffic at this time.

Of primary significance in connection with the roadometer was a finding that the user rating of a pavement can be estimated closely from a series of physical measurements, including the measurement of wheelpath roughness. Other items found to influence rating significantly were cracking, patching, and rutting. Mathematical expressions involving these items have been developed to show the relationship between the ratings applied by rating items and the objective physical measurements. The estimate of the serviceability index obtained from the use of these mathematical expressions is called the "present serviceability index."

One such mathematical expression was developed for portland cement concrete pavements and another for bituminous concrete surfaces. Measured values of roughness, made with a BPR-type roadometer or other devices, together with measurements of cracking and patching, and of rutting in the case of bituminous surfaces, can be substituted in these equations to determine the serviceability index at any time.

The original mathematical expressions that were developed at the AASHO Road Test take into account the measurement of surface irregularities made by a profilometer which records variances in the longitudinal slope of pavements. A first step in using the Illinois roadometer in connection with determining the present serviceability index of pavements was its correlation with the AASHO device. This correlation furnished the following two equations for pavement serviceability based on the Illinois roadometer results:

Portland Cement Concrete (Rigid) Pavement

$$PSI = 12.0 - 4.27 \log RI - 0.09 \sqrt{C+P}$$

in which

PSI = present serviceability index;
RI = roughness index (by Illinois roadometer);
C = lineal feet of crack per 1,000 sq ft of pavement surface; and
P = square feet of bituminous patching per 1,000 sq ft of pavement surface.

Bituminous Concrete (Flexible) Pavement

$$PSI = 10.91 - 3.90 \log RI - 0.01\sqrt{C+P} - 1.38D^2$$

in which

PSI = present serviceability index,
RI = roughness index (by Illinois roadometer);
C = square feet of cracked area per 1,000 sq ft of pavement surface;
P = square feet of patching per 1,000 sq ft of pavement surface; and
D = average rut depth in inches at deepest part of rut.

The development of the present serviceability concept has enhanced greatly the value of the various devices used to determine the riding quality of pavements, including the BPR-type roadometer. As discussed later, it has provided a means for establishing a working relationship between roadometer measurements and highway user opinion.

The importance of the present serviceability concept should not be underestimated. This concept offers the engineer, for the first time, a good possibility for designing a pavement to be so constructed that it will carry a specified number of load applications until some predetermined terminal serviceability index is reached.

Also through the use of this concept, there is the possibility that the engineer can rate existing pavements presently in service and predict with reasonable accuracy the number of axle loadings that may be carried in the future before some predetermined terminal serviceability index is reached.

Knowledge of the serviceability level at which it is desirable to reconstruct or resurface pavements (terminal serviceability) is necessary for applying the present serviceability concept in design and in assessing the remaining useful life of pavements. To provide this information, the Illinois Division of Highways is currently engaged in making an intensive study of pavements scheduled for retirement. Roughness recordings are being made in all parts of the State on pavements scheduled for reconstruction or resurfacing because of structural inadequacies. Surveys of patching and cracking (and rutting of bituminous surfaces) are being made in conjunction with the roadometer studies to provide values for substitution in the terms of present serviceability equations.

Preliminary results of the roadometer and other measurements on pavement scheduled for retirement in Illinois because of structural inadequacies have been encouraging in that they have yielded reasonable values of the terminal serviceability index. Values thus far have ranged from about 2.5 to less than 1.0. The lower values of the terminal serviceability index have been found to be consistently associated with pavements carrying a relatively small volume of traffic. High-volume pavements are more frequently retired at the higher index values. This phase of the study is not complete and is continuing.

## ADJECTIVE RATINGS OF PAVEMENT RIDING QUALITY

A second important application of the present serviceability concept has been the establishment of a working relation between roadometer measurements and highway user opinion. Heretofore, the usefulness of the roadometer has been somewhat limited because of the lack of knowledge concerning this relationship.

In the assessment of highway user opinion that led to the development of the mathematical expressions under the present serviceability concept, adjective descriptions of user opinion were applied to various ranges of the rating scale of 0 to 5. The removal of the terms of the present serviceability equation relating to cracking, patching, and rutting allows a determination of the influence of the roughness index alone on the present serviceability index. For the Illinois roadometer, this provided the following group descriptions:

| AASHO Present Serviceability Rating | | Illinois Roadometer Roughness Index | | |
|---|---|---|---|---|
| Numerical | Adjective | Rigid Pavement (in./mi) | Flexible Pavement (in./mi) | Adjective Rating |
| 5 | | 45 | 35 | - |
| 4 | Very good | 75 | 60 | Very smooth |
| | Good | 90 | 75 | Smooth |
| 3 | | 125 | 105 | Slightly rough |
| | Fair | 170 | 145 | Rough |
| 2 | | 220 | 190 | Very rough |
| 1 | Poor | 375 | 330 | Unsatisfactory |
| 0 | Very poor | | | - |

The adjective ratings at the right of the preceding table are being applied tentatively by Illinois to further subdivide the groupings established by use of the present serviceability equations.

In the table, the response of the device to irregularities in portland cement concrete and bituminous concrete surfaces is apparently somewhat different.

Roughness indices have been determined in Illinois for many hundreds of miles of portland cement concrete pavements and bituminous concrete resurfacings. The roughness indices have covered a sufficient range to indicate beyond doubt that they are truly definitive of the riding quality of pavement surfaces. Of considerable significance from a correlative standpoint has been the fact that the isolated complaints that have been received about the riding condition of new pavements invariably have concerned pavements that recordings have shown to be rough. Fortunately, the mileage of such pavement has been small.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

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