B PSR, PSI, AASHO Road Test, and ESAL Concept

B.1 PSR, PSI Concept

資料來源：

Carey, W. N., and P. E. Irick, "The Pavement Serviceability-Performance Concept," Highway Research Board, Bulletin No, 250, 1960.

B.2 AASHO Road Test

資料來源：

Highway Research Board, "The AASHO Road Test," Report 5, Pavement Research, Special Report 61E, Publication No. 954, National Research Council, Washington, D.C., 1962.

B.3 ESAL Concept and Calculations

資料來源：

1. Highway Research Board, "The AASHO Road Test," Report 5, Pavement Research, Special Report 61E, Publication No. 954, National Research Council, Washington, D.C., 1962.
2. 中華民國交通部臺灣區國道高速公路局，「高速公路年報」，中華民國八十五年。

The Pavement Serviceability-Performance Concept

Ref.: Highway Research Board Bulletin No. 250 (1960)

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A system is described wherein the serviceability of pavements is rated subjectively by a panel made up of men selected to represent many important groups of highway users. Through multiple regression analysis a mathematical index is derived and validated through which pavement ratings can be satisfactorily estimated from objective measurements taken on the pavements. These serviceability indices (or the direct ratings) always refer to the conditions existing at the time the measurements (or ratings) are made. Performance of a pavement may then be determined by summarizing the serviceability record over a period of time.

The system, developed at the AASHO Road Test, has potential for wide application in the highway field, particularly in sufficiency rating, evaluation of design systems, and evaluation of paving materials and construction techniques through the provision of an objective means for evaluation of performance.

● THE RELATIVE PERFORMANCE of various pavements is their relative ability to serve traffic over a period of time. There have been no widely accepted definitions of performance that could be used in the evaluation of various pavements or that could be considered in the design of pavements. In fact, design systems in general use in highway departments do not include consideration of the level of performance desired. Design engineers vary widely in their concepts of desirable performance. By way of example, suppose that two designers were given the task of designing a pavement of certain materials for certain traffic and environment for 20 years. The first might consider his job to be properly done if not a single crack occurred in 20 years, whereas the second might be satisfied if the last truck that was able to get over the pavement made its trip 20 years from the date of construction. There is nothing in existing design manuals to suggest that either man was wrong. This is simply to demonstrate that any design system should include consideration of the level of serviceability to traffic that must be maintained over the life of the road. How long must it remain smooth and how smooth?

One popular design system involves determination of the thickness of slab required to hold certain computed stresses below a certain level. It is clear that cracks will occur if a pavement is overstressed, but nowhere can be found any reference to the effect of such cracks on the serviceability of the pavement. Engineers will agree that cracks are undesirable, and that they require maintenance, but the degree of undesirability seems to have been left dimensionless. It may be apparent that one pavement has performed its function of serving traffic better than another, but a rational answer to the question, "How much better?" has not been available.

To provide dimensions for the term "performance" a system has been devised that is rational and free from the likelihood of bias due to the strong personal opinions of groups or individuals. It is easily conceivable that such a system could be adopted by all departments, thus providing for the first time a national standard system for rating highways and pavements.

Before discussing the derivation and a particular application of the pavement serviceability-performance system, it is necessary to set down some fundamental assumptions upon which the system is based.

1. There is a statement attributed to D.C. Greer, State Highway Engineer of Texas, that "highways are for the comfort and convenience of the traveling public." A reasonable inference from this simple statement is that the only valid reason for any road or highway is to serve the highway users. Another definitive opinion is that "a good highway is one that is safe and smooth."

2. The opinion of a user as to how he is being served by a highway is by-and-large subjective. There is no instrument that can be plugged into a highway to tell in objective units how well it is serving the users. The measurement of damage to goods attributed to rough roads may provide an exception to this rule (but one of minor importance), as a road rough enough to damage properly packed and properly suspended goods would be classed subjectively so low by all users that little could be gained by an objective measure.

3. There are, however, characteristics of highways that can be measured objectively which, when properly weighted and combined, are in fact related to the users subjective evaluation of the ability of the highway to serve him.

4. The serviceability of a given highway may be expressed by the mean evaluation given it by all highway users. There are honest differences of opinion, even among experts making subjective evaluations of almost anything. Thus, there are differences of opinion as to which automobile in a given price range is best; differences among judges of a beauty contest; differences as to which bank, broker, grocery store, or bar to patronize; etc. Opinion as to the serviceability of highways is no exception. Economic considerations alone cannot explain these differences.

Thus, in order for normal differences of opinion to be allowed with the smallest average error for each individual highway user, serviceability, as previously stated, may be expressed in terms of the mean evaluation of all users.

5. Performance is assumed to be an over-all appraisal of the serviceability history of a pavement. Thus it is assumed that the performance of a pavement can be described if one can observe its serviceability from the time it was built up to the time its performance evaluation is desired.

AN EXAMPLE OF THE USE OF THE SERVICEABILITY-PERFORMANCE SYSTEM

In this section is described a typical example of the system which has been in actual field use at the AASHO Road Test. Definitions and detailed steps in the development and use of a Performance Index for evaluation of the Road Test pavements are included. It is emphasized that the case herein described is only one of many possible applications of the principles involved. It happened to relate to the performance of the pavements only, yet it would have been easy to extend the system to provide a measure of the sufficiency of the entire highway, including grade, alignment, access, condition of shoulders, drainage, etc., as well as characteristics of the pavement itself.

Purpose

The principle objective for the AASHO Road Test calls for significant relationships between performance under specified traffic and the design of the structure of certain pavements. To fulfill this objective, an adequate and unambiguous definition of pavement performance was required. For reasons previously mentioned none was available.

Special Considerations

In addition to the primary assumptions listed in the early paragraphs of this report, certain special considerations relating to the specific requirements of the Road Test were included.

Inasmuch as the project was designed to provide information relating to the pavement structure only, certain aspects of normal pavement serviceability were excluded from consideration, including surface friction, condition of shoulders, etc.

Test sections at the Road Test were as short as 100 ft, too short for a satisfactory subjective evaluation of their ability to serve traffic (most highway users consider a high-speed ride over a pavement necessary before they will rate it). Thus, objective measurements that could be made on the short sections had to be selected and used in such a way that pavements only 100 ft long could be evaluated as though they were much longer.

Definitions

To fulfill the requirements of the Road Test, rather ordinary terms were given specific definitions as follows:

Present Serviceability—the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic in its existing condition. (Note that the definition applies to the existing condition—that is, on the date of rating—not to the assumed condition the next day or at any future or past date.) Although this definition applies to the Road Test and may apply to any primary highway system, the system could easily be modified for use with city streets, farm roads, etc. Obviously, serviceability must be defined relative to the intended use of the road.

Individual Present Serviceability Rating—an independent rating by an individual of the present serviceability of a specific section of roadway made by marking the appropriate point on a scale on a special form (Fig. 1). This form also includes provision for the rater to indicate whether or not the pavement is acceptable as a primary highway. For the Road Test application, when rating highways other than those in the primary system, the rater was instructed to exclude from consideration all features not related to the pavement itself, such as right-of-way width, grade, alignment, shoulder and ditch condition, etc.

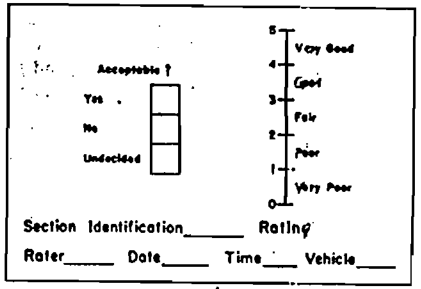

Figure 1. Individual present serviceability rating form.

Present Serviceability Rating (hereafter PSR)—the mean of the individual ratings made by the members of a specific panel of men selected for the purpose by the Highway Research Board. This panel was intended to represent all highway users. It included experienced men, long associated with highways, representing a wide variety of interests, such as highway administration, highway maintenance, a federal highway agency, highway materials supply (cement and asphalt), trucking, highway education, automotive manufacture, highway design, and highway research.

Present Serviceability Index (hereafter PSI)—a mathematical combination of values obtained from certain physical measurements of a large number of pavements so formulated as to predict the PSR for those pavements within prescribed limits.

Performance Index (hereafter PI)—a summary of PSI values over a period of time. There are many possible ways in which the summary value can be computed. Perhaps the simplest summary consists of the mean ordinate of the curve of PSI against time.

Steps in Formulation of a Present Serviceability Index

The following represents a minimum program for the establishment, derivation and validation of a PSI (or any similar index that may be considered for another purpose).

1. Establishment of Definitions—There must be clear understanding and agreement among all those involved in rating and in formulation and use of the index as to the precise meanings of the terms used (see preceding definitions for Road Test case). Exactly what is to be rated, what should be included, and what excluded from consideration?

2. Establishment of Rating Group or Panel—Because the system depends primarily on the subjective ratings of individuals, great care should be taken in the selection of the persons who will make up the rating group. Inasmuch as serviceability is here defined to be the mean opinion of this group, it is important that the raters represent highway users. They should be selected from various segments of the users with divergent views and attitudes.

3. Orientation and Training the Rating Panel—An important step is that in which the members of the Panel are instructed in the part they are to play. They must clearly understand the pertinent definitions and the rules of the game. It has been found worthwhile to conduct practice rating sessions where the raters can discuss their ratings among themselves. Note that when they make their official ratings they must work independently, with no opportunity for discussion of the ratings until the entire session has been completed.

4. Selection of Pavements for Rating—Ratings are to be made of the serviceability of pavements; therefore, a wide range of serviceability should be represented among the pavements that are selected for rating. Moreover, represented among the sections selected should be pavements containing all of the various types and degrees of pavement distress that are likely to influence the serviceability of highways. Prior to a field rating session, engineers study the highway network in the area under consideration (say 200 mi or less in diameter) and pick sections of roadway such that a reasonable balance is obtained among sections, of which some are obviously in very good condition, some are good, some fair, some poor and some obviously very poor. The Road Test system was based on four rating sessions in three different states in which 138 sections of pavement were studied. About one-half were flexible pavement and one-half rigid. The Road Test Panel members agreed among themselves that the minimum desirable length of a pavement to be rated was 1,200 ft; however, in a few cases shorter sections were included. This length was sufficient so that the raters could ride over the section at high speed and not be influenced by the condition of pavement at either end of the section.

5. Field Rating—The members of the Panel are taken in small groups to the sections to be rated. They are permitted to ride over each section in a vehicle of their choice (usually one with which they are familiar), to walk the pavement and to examine it as they wish. Each rater works independently—there is no discussion among the raters. When each is satisfied as to his rating, he marks his rating card and turns it in to a staff representative. The group then moves on to the next section. Each group takes a different route in order to reduce the possibility of bias over the day (raters may rate differently in the afternoon than in the morning; therefore, the groups are scheduled so that some sections are rated by one or two groups in the morning and the same sections by the other groups in the afternoon). It has been found that, near metropolitan areas, sections with satisfactorily different characteristics can be found near enough together so that the raters can travel routes containing about 20 sections per day. When rating present serviceability of a pavement, raters have found it helpful to ask themselves: "How well would this road serve me if I were to drive my own car over roads just like it all day long today?" Here again, of course, serviceability is related to the intended use of the road (primary highway, city street, farm road, etc.).

6. Replication—It is necessary to determine the ability of the Panel to be consistent in its ratings. The Road Test Panel rated many sections twice, first on one day and again on another day near enough to the first so that the section did not change physically, yet remote enough so that all extraneous influences on the raters would be in effect. In general it might be expected that replicate ratings would differ more when separated by several months than when separated by only one day. For this reason it may be supposed that the replication differences observed in the Road Test Rating sessions are to some degree an underestimate of replication differences in a larger time reference frame. The difference between repeated ratings on the same section is a criterion for the adequacy of a present serviceability index derived from measurements.

7. Validation of Rating Panel—Because the Panel is intended to represent all highway users, it is necessary to test its ability to do so. To a limited extent such validation was obtained for the Road Test Panel by selecting other groups of users and having them rate some of the same sections that had been rated by the Panel. One such group consisted of two professional commercial truck drivers who made their ratings

based on the rides they obtained when driving their own fully-loaded tractor-semitrailer vehicles. Another group was made up of ordinary automobile drivers not professionally associated with highways. For the sections involved these studies indicated that the ratings given pavements by the Road Test Panel were quite similar to those that were given by the other user groups. Of course, if a greater number of sample groups had been studied, more positive statements could be made as to how well the Panel represents the universe of all users.

8. Physical Measurements—If it is practicable for the Panel to rate all roads in the area of interest often enough, no measurements need be taken. Analyses may be based on the PSR itself. Since it was not possible for the Panel to rate the Road Test sections (ratings were desired every two weeks), it was necessary to establish a PSI or index that would predict the Panel's ratings. To accomplish this, measurements of certain physical characteristics of the pavements were necessary. In order to determine which measurements might be most useful, the members of the Panel were asked to indicate on their rating cards which measureable features of the roadway influenced their ratings. This study made it apparent that present serviceability was a function primarily of longitudinal and transverse profile, with some likelihood that cracking, patching, and faulting would contribute. Thus, all of these characteristics were measured at each of the 138 sections in three states that were rated by the Panel. It should be noted that several other objective measurements could have been added to the list if other phenomena were permitted consideration by the established rules of the game. In this category might be skid resistance, noise under tires, shoulder and ditch conditions, etc.

Measurements fall rather naturally into two categories: those that describe surface deformation and those that describe surface deterioration. Of course, phenomena in the second category may or may not influence measurements in the first category. Measures of surface deformation will reflect the nature of longitudinal and transverse profiles—or may represent the response of a vehicle to the profile, as does the BPR roughometer. Supplemental profile characteristics, such as faulting, will ordinarily be measured. Present and past surface deterioration will be reflected through measures of cracking, spalling, potholing, patching, etc., and may include phenomena whose influence on present serviceability ratings range from negligible to appreciable.

9. Summaries of Measurements—There are many different ways to summarize longitudinal and transverse profiles. For example, longitudinal profile may be expressed as total deviation of the record from some baseline in inches per mile, number of bumps greater than some minimum, some combination of both of these, or by any number of other summary statistics involving variance of the record, power spectral density analysis, etc. Transverse profile may be summarized by mean rut depth, variance of transverse profile, etc. The variance of rut depth along the wheel paths is also a useful statistic. Cracking occurs in different classes of severity, as do other measures of surface deterioration, and measurements in any of these classes may be expressed in one unit or another.

10. Derivation of a Present Serviceability Index—After having obtained PSR's and measurement summaries for a selection of pavements, the final step is to combine the measurement variables into a formula that "gives back" or predicts the PSR's to a satisfactory approximation. Part of this procedure should consist in determining which of the measurement summaries have the most predictive value and which are negligible after the critical measurements are taken into account. The technique of multiple linear regression analysis may be used to arrive at the formula, or index, as well as to decide which measurements may be neglected. For example, it can turn out that a longitudinal profile summary will be sensitive to faulting so that faulting measurements need not appear in the index formula whenever this profile measure is included.

The decisions as to which terms should be in the serviceability formula and which terms should be neglected may be made by comparing the lack of success with which the formula "gives back" the ratings with a preselected criterion for closeness of fit—such as the Panel's replication error (see previous discussion, item 5). That is, there is no justification for a formula that can predict a particular set of ratings with greater precision than the demonstrated ability of the Panel to give the same ratings to the same pavements twice. Thus the multiple linear regression analysis will yield a formula that

will combine certain objective measurements to produce estimates of the Panel's ratings to an average accuracy no greater than the Panels's average ability to repeat itself.

Performance

In the preceding section the steps in the formulation of a present serviceability index were delineated. The index is computed from a formula containing terms related to objective measurements that may be made on any section of highway at any time. At the AASHO Road Test these measurements are made and the index computed for each test section every two weeks. Thus a serviceability-time history is available for each test section beginning with the time test traffic operation was started. As can be seen from Figure 1, the present serviceability values range in numerical value from zero to five.

In order to fulfill the first Road Test objective, to find relationships between performance and pavement structure design, some summarization of the serviceability-time history is implied. Performance may be said to be related to the ability of the pavement to serve traffic over a period of time. A pavement with a low serviceability during much of its life would not have performed its function of serving traffic as well as one that had high serviceability during most of its life, even if both ultimately reached the same state of repair.

The Road Test staff studied many alternate techniques for summarizing the serviceability-time history into an index of performance. The performance index chosen consisted of the mean ordinate of the serviceability-time history record. The choice of mean ordinate of serviceability-time record was largely due to its simplicity and the ease with which it can be understood by those interested in the Road Test findings.

ROAD TEST INDEXES

The techniques previously described were used in the derivation of present serviceability indexes for the AASHO Road Test. This section of the report includes tabulations of the actual data obtained in the field rating sessions by the Road Test Rating Panel and data obtained from the objective measurements of the pavements rated. Relationships among the ratings and various measurements are shown graphically and the results of the regression analyses in which the serviceability indexes were derived are given.

The matter of precision required of an index and precision attained in the Road Test indexes is discussed. Alternate measurement systems are mentioned for the benefit of agencies not able to equip themselves with elaborate instruments.

Ratings for Selected Pavements

After the establishment of concepts, ground rules, and rating forms for present serviceability ratings, the AASHO Road Test Performance Rating Panel rated 19 pavement sections near Ottawa, Ill., on April 15-18, 1958; 40 sections near St. Paul-Minneapolis on August 14-16, 1958; 40 sections near Indianapolis on May 21-23, 1959; and 39 sections on and near the Road Test in Illinois on January 20-22, 1960. Ten Illinois sections, 20 Minnesota sections, 20 Indiana sections and 24 sections on and near the Road Test were flexible pavements, whereas all remaining sections were rigid pavements. Each section was 1,200 ft long except those on the Road Test, which averaged 215 ft. With the generous cooperation of the respective state highway departments, sections at each location were selected to represent a wide range of pavement conditions.

Coincident with the rating session, Road Test crews and instruments were used to obtain condition surveys and profile measurements for each section. Summaries for all evaluations of the 74 flexible pavement sections are shown in Table 1; corresponding evaluations for the first 49 rigid pavements are given in Table 2.

The principal objective of the fourth rating session was to rate flexible pavement sections that included rather severe degrees of rutting—a phenomenon not included in the previous sets of flexible pavement. A second objective of the fourth session was to rate a small number of rigid pavements only for the purpose of checking present serv-

TABLE 1

DATA FOR 74 SELECTED FLEXIBLE PAVEMENTS

Pvt. Loc.	Sect. Code	Present Serviceability Ratings: AASHO Panel: 1st PSR	AASHO Panel: Replic. diff in PSR	AASHO Panel: Std. dev. of PSR among raters	Truck Drvrs: PSR	Canad. Raters: PSR	Acceptability Opinions: AASHO Panel Fraction: Yes	Fraction: No	$\overline{SV}$ Mean Slope Var'nce in Wh'pths ($x10^6$)	$\overline{AR}$ Mean AASHO Rdm'tr in Wh'pths in./mi. @10 mph	$\overline{RD}$ Mean Rut Depth, (in.)	$\overline{RDV}$ Mean Rut Depth Var'nce $in^2 x 100$	Major Cracking: Class 2 + Class 3, ft^2/1000ft^2	Long. & Trans. ft./1000ft^2	Patching: P Ft^2 per 1000 ft^2	Transformations: Log (1+$\overline{SV}$)	$\overline{RD}^2$	Sq. rt. C+P	PSI 121 Pres. Serv. Index	Resid. Diff. Betw'n PSR & PSI
Ill.	F 3	4.3		.1	4.5	4.3	1.0	.0	2.8		.10	.7	0	0	0	.57	.01	.0	3.9	.4
	F 4	2.4		.4		2.0	.0	.6	20.5		.22	9.2	343	0	0	1.33	.05	18.5	2.3	.1
	F 5	3.3		.7	3.5	2.6	.6	.2	9.2		.08	3.6	8	0	0	1.01	.01	2.8	3.1	.2
	F 6	4.4	.1	.2	3.5		1.0	.0	3.5		.08	.7	0	0	0	.65	.00	.0	3.8	.6
	F 7	3.8	.1	.6	2.5	3.6	.9	.0	15.5		.06	.4	0	0	0	1.22	.00	.0	2.7	1.1
	F 8	2.6	.2	.7	2.0	2.7	.3	.6	9.5		.08	5.7	64	0	0	1.02	.01	8.0	3.0	.4
	F 9	3.2	.2	.6	3.0	3.0	.6	.2	14.0		.15	3.4	2	0	3	1.18	.02	2.2	2.7	.5
	F10	2.4	.0	.5	3.0	2.2	.1	.6	16.8		.16	3.4	17	0	14	1.25	.03	5.6	2.6	.2
	F11	1.3	.3	.5	1.5		.0	1.0	42.8		.24	10.3	292	0	11	1.64	.07	17.4	1.6	.3
	F12	1.1	.3	.2	1.0	1.7	.0	1.0	56.0		.19	10.9	21	0	2	1.76	.04	4.8	1.6	.5
Minn.	101	3.8	.3	.4			1.0	.0	1.9		.04	.4	0	29	0	.46	.00	5.4	4.1	.3
	102	3.8		.6			1.0	.0	1.5		.09	.3	0	34	0	.40	.00	5.8	4.2	.4
	103	3.8		.4			1.0	.0	1.7		.05	.2	0	14	0	.43	.00	3.7	4.2	.4
	104	3.8	.0	.4			1.0	.0	2.1		.04	.4	0	9	0	.49	.00	3.0	4.1	.3
	105	3.2		.4			.6	.1	7.0		.14	.5	0	0	10	.90	.02	3.2	3.3	.1
	106	1.3		.4			.0	1.0	58.5		.07	6.6	145	22	35	1.77	.00	14.2	1.5	.2
	107	1.3	.2	.4			.0	1.0	58.4		.08	2.9	75	12	55	1.77	.01	11.9	1.5	.2
	108	2.1		.6			.0	.9	17.6		.13	3.2	15	0	5	1.27	.03	4.5	2.5	.4
	109	1.5	.2	.2			.0	1.0	36.2		.37	5.6	30	2	76	1.57	.13	10.4	1.7	.2
	110	2.4	.0	.9			.0	.8	11.4		.07	3.2	2	0	3	1.09	.00	2.2	2.9	.5
	111	4.2		.2			1.0	.0	1.7		.11	.2	0	0	0	.42	.01	.0	4.2	.0
	112	3.9		.4			1.0	.0	1.4		.09	.2	0	0	0	.38	.01	.0	4.3	.4
	113	3.1	.1	.6			.4	.4	7.8		.08	1.3	1	68	66	.94	.01	11.6	3.1	.0
	114	2.2		.7			.0	.9	27.8		.13	5.2	0	1	4	1.46	.02	2.2	2.2	.0
	115	1.5		.6			.0	1.0	33.4		.08	5.4	0	7	6	1.54	.01	3.6	2.0	.5
	116	2.9	.5	[illegible]			.6	.1	6.0		.02	.7	0	180	0	.85	.00	13.4	3.3	.4
	117	1.6	.0	.6			.0	1.0	39.4		.12	4.6	0	0	0	1.61	.01	.0	1.9	.3
	118	4.0	.2	.3			1.0	.0	1.6		.04	.2	0	0	0	.41	.00	.0	4.2	.2
	119	4.2		.5			1.0	.0	1.3		.03	.2	0	0	0	.36	.00	.0	4.3	.1
	120	2.9	.1	.5			.3	.2	5.8		.01	1.4	0	74	0	.84	.00	8.6	3.3	.4
	301	4.1		.6			1.0	.0	4.6	101	.24	.4	44	43	0	.75	.06	9.3	3.4	.7
	302	4.0		.5			1.0	.0	5.4	123	.34	.5	204	75	0	.81	.12	16.7	3.2	.8
Ind.	303	3.2		.5			.6	.1	20.1	126	.02	.4	12	1	0	1.32	.00	3.6	2.5	.7
	304	2.4	.0	.4			.2	.5	29.2	134	.17	1.8	455	0	17	1.48	.03	21.7	2.0	.4
	305	2.9	.1	.3			.4	.3	9.1	129	.10	1.2	292	0	32	1.00	.01	18.0	2.9	.0
	306	2.4	.2	.5			.3	.5	20.5	161	.12	2.4	816	0	0	1.33	.01	28.6	2.2	.2
	307	1.7	.3	.6			.1	.9	95.9	383	.02	1.9	719	0	111	1.99	.00	28.8	1.0	.7
	308	1.0		.4			.0	1.0	51.8	296	.03	5.1	691	0	161	1.72	.00	29.2	1.5	.5
	309	1.3		.4			.0	1.0	41.2	233	.14	7.0	613	0	159	1.62	.02	27.8	1.6	.3
	310	3.2		.6			.7	.1	11.5	144	.18	2.0	17	0	0	1.10	.03	4.1	2.8	.4
	311	2.7		.4			.4	.4	15.0	162	.14	.8	45	0	0	1.20	.02	6.7	2.6	.1
	312	1.6		.4			.0	1.0	49.8	217	.23	2.3	502	0	31	1.71	.05	23.1	1.5	.1
	313	1.4		.4			.0	1.0	42.0	182	.27	2.9	437	0	72	1.63	.07	22.6	1.7	.3
	314	2.6		.5			.3	.4	19.0	127	.24	1.2	10	64	2	1.30	.06	8.7	2.4	.2
	315	3.4		.7			.9	.0	6.9	107	.22	.2	183	46	0	.90	.05	15.1	3.1	.3
	316	2.9		.5			.5	.4	11.3	140	.09	.8	177	0	4	1.09	.01	13.4	2.8	.1
	317	4.3		.2			1.0	.0	2.9	95	.01	.1	0	0	0	.59	.00	.0	3.9	.4
	318	4.3		.3			1.0	.0	3.3	92	.00	.0	1	0	0	.63	.00	1.0	3.8	.5
	319	4.2	.0	.3			1.0	.0	3.8	92	.12	.2	0	1	0	.68	.01	1.0	3.7	.5
	320	3.9	.3	.4			.9	.0	3.8	105	.16	.4	0	2	0	.68	.03	1.4	3.7	.2
Test Road Sect.	501	3.8		.3			1.0	.0	5.8	132	.08	.3	0	0	0	.83	.01	.0	3.4	.4
	502	3.4		.6			.8	.1	10.3	168	.20	2.2	51	0	0	1.05	.04	7.2	2.9	.5
	503	3.1	.0	.3			.7	.0	7.6	129	.11	.8	17	0	7	.93	.01	4.9	3.2	.1
	504	4.1		.2			1.0	.0	2.6	109	.03	.2	0	0	0	.56	.00	.0	4.0	.1
	505	3.4		.2			1.0	.0	3.8	89	.33	.9	14	0	0	.68	.11	3.7	3.5	.1
	506	3.4		.4			.9	.0	4.8	89	.24	.9	0	0	0	.76	.06	.0	3.5	.1
	507	2.8		.4			.6	.3	7.6	103	.43	1.4	5	0	25	.93	.18	5.4	2.9	.1
	508	3.5		.5			.9	.0	4.0	75	.46	.5	0	0	0	.70	.21	.0	3.4	.1
	509	3.3		.5			.8	.0	2.9	84	.39	.4	9	0	0	.59	.15	3.0	3.7	.4
	510	3.3		.4			.9	.0	5.0	90	.44	1.0	2	0	0	.78	.19	1.3	3.3	.0
	511	3.6		.5			1.0	.0	3.5	90	.47	.6	0	0	0	.65	.22	.0	3.5	.1
	512	3.2		.7			.8	.0	5.1	87	.53	1.3	0	0	0	.79	.28	.0	3.1	.1
	513	3.4		.5			.8	.0	2.6	75	.56	.5	0	0	0	.56	.31	.0	3.5	.1
	514	1.8		.5			.0	.9	11.1	122	.73	5.1	80	0	16	1.08	.53	9.8	2.1	.3
	515	3.3		.7			.9	.0	2.5	79	.38	.5	0	0	0	.54	.14	.0	3.3	.5
	516	2.6		.6			.4	.3	5.4	86	.55	.6	15	0	0	.81	.30	3.9	3.0	.4
	517	3.2		.6			.5	.1	5.4	83	.54	.7	1	0	0	.81	.29	1.2	3.1	.1
	518	1.7		.4			.0	.8	21.0	149	.92	2.8	222	0	0	1.34	.85	14.9	1.2	.5
	519	2.4		.5			.1	.4	6.5	99	.53	1.5	21	0	0	.88	.28	4.6	2.9	.5
	520	3.0	.2	.6			.6	.1	6.8	89	.46	1.1	0	0	0	.89	.21	.0	3.0	.0
Off Site Sect.	521	3.3		.5			.7	.1	4.3	118	.09	.4	0	0	0	.72	.01	.0	3.6	.3
	522	2.7		.4			.6	.1	13.7	185	.11	3.8	300	0	1	1.17	.01	17.4	2.6	.1
	523	2.4		.4			.2	.2	10.8	137	.22	1.3	496	0	52	1.07	.05	23.4	2.7	.3
	524	0.9		.5			.0	1.0	88.1	281	.25	6.2	392	0	60	1.95	.06	21.2	1.0	.1
Sum		215.4	3.9	34.2												75.19	5.59	565.7	215.4*	22.3
Mean		2.91	.16	.46												1.02	.076	7.64	2.91	.30
Sum of Squares		66.85														13.27	1.34	5255	56.42*	10.42*

	Log (1+$\overline{SV}$)	$\overline{RD}^2$	Sq. rt. C+P
Sum of Products with PSR	-26.69	-1.51	-369.3
Sum of Products with log(1+$\overline{SV}$)		-.166	171.63
Sum of Products with $\overline{RD}^2$			-3.90

*Obtained from Unrounded Calculations

$$PSI\ 121 = 5.03 - 1.91 \log(1+\overline{SV}) - 1.38\overline{RD}^2 - .01\sqrt{C+P}$$

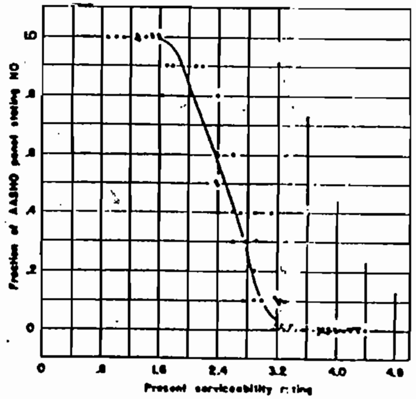

Figure 4. Unacceptability vs present serviceability rating; 74 flexible pavements.

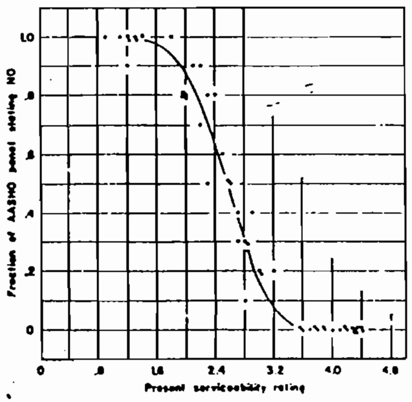

Figure 5. Unacceptability vs present serviceability rating; 49 rigid pavements.

The symbol $\overline{SV}$ is used for the summary statistic of wheelpath roughness as measured by the Road Test longitudinal profilometer. For each wheelpath the profilometer produces a continuous record of the pavement slope between points 9 in. apart. For a particular wheelpath, the slopes are sampled, generally at 1-ft intervals, over the length of the record. A variance[1] is calculated for the sample slopes in each wheelpath, then the two wheelpath slope variances are averaged to give $\overline{SV}$.

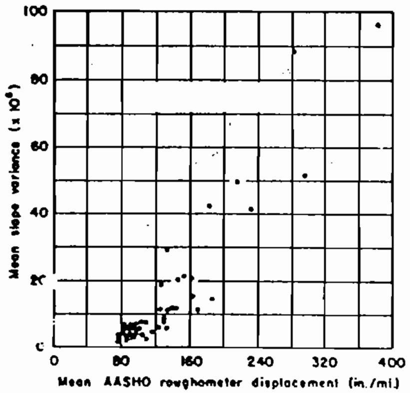

Figure 6. Slope variance vs AASHO roughometer displacement; 44 flexible pavements.

A Bureau of Public Roads road roughness indicator, or roughometer, has been adapted for use at the AASHO Road Test, but this development was not made until just prior to the Indiana rating session and still more developmental work has been done on the AASHO roughometer since the Indiana session. The AASHO roughometer has a modified output and is run at 10 mph, so roughometer values shown in Tables 1 and 2 are not those that would be obtained with the BPR roughometer at 20 mph. Nevertheless, roughometer values in inches per mile are given in the tables so that it may be noted that the roughometer values averaged for both wheelpaths, $\overline{AR}$, are correlated with the corresponding mean slope variances. Figures 6 and 7 show the extent of this correlation for the last two rating sessions.

One other instrument, a rut depth gage, was used to obtain profile characteristics of the flexible pavement sections. This gage is used to determine the differential elevation between the wheelpath and a line connecting two points each 2 ft away (transversely) from the center of the wheelpath. Rut depth measurements were obtained at 20-ft intervals in both wheelpaths. Average rut depth values, $\overline{RD}$, for the flexible sections are given in Table 1, where it may be noted that the values range from 0 to nearly 1 in. Variances were calculated for the rut depths in each wheelpath, then the two wheelpath variances were averaged to give the $\overline{RDV}$ values given in Table 1. Figure 8 indicates the correlation between $\overline{SV}$ and $\overline{RDV}$ for the 74 flexible sections.

[1]The variance of a set of N sample values, $Y_1, Y_2, \ldots, Y_N$ is defined to be the sum of all N squared deviations from the mean divided by N-1. Thus the variance of Y is $\sum (Y-\overline{Y})^2/(N-1)$, where $\overline{Y} = \sum Y/N$ is the sample mean.

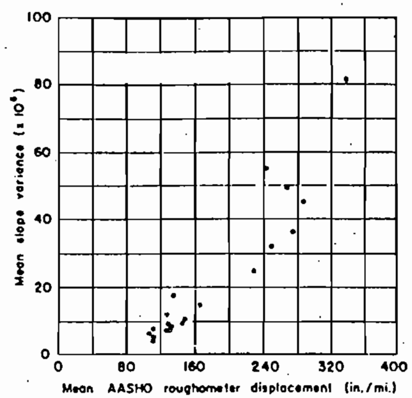

Figure 7. Slope variance vs AASHO roughometer displacement; 20 rigid pavements.

Profile information for rigid pavements included a measure of faulting in the wheelpaths. These measurements are given in Table 2, expressed in total inches of faulting (in wheelpaths only) per 1,000 ft of wheelpath.

The remaining measurements for flexible pavement sections are given in Table 1 under the headings of area affected by class 2 and class 3 cracking, length of transverse and longitudinal cracks, and patched area, where areas and lengths are expressed per 1,000 sq ft of pavement area. Corresponding measurements for rigid pavements are shown in Table 2 in terms of length of class 2 and sealed cracks, spalled area, and patched area. Lengths for rigid pavement cracks were determined by projecting the cracks both transversely and longitudinally, choosing the larger projection, then expressing the accumulated result in feet per 1,000 sq ft of pavement area. Only spalled areas having diameters greater than 3 in. were considered, and both spalling and patching are expressed in square feet per 1,000 sq ft of pavement area. Virtually any pair of measurements are intercorrelated to some degree, some more highly than others. Figures 9 and 10 indicate the degree to which $\overline{SV}$ is correlated with the sum of cracking and patching values. It is obvious that a stronger correlation exists in Figure 10 than in Figure 9. If either correlation were perfect, one or the other of the plotted variables would be redundant in an index of present serviceability.

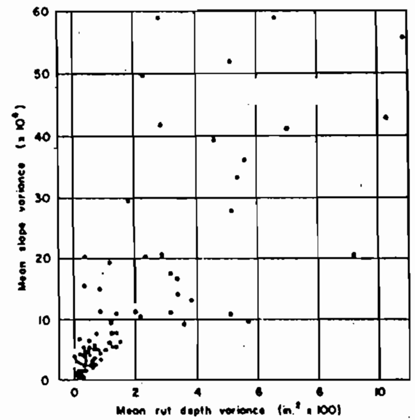

Figure 8. Rut depth variance vs slope variance; 74 flexible pavements.

The remaining columns in Tables 1 and 2 are connected with the development of present serviceability indices and will be discussed in succeeding paragraphs.

Hypothesis and Assumptions for Present Serviceability Index

It has been stated that one requirement for an index of present serviceability is that when pavement measurements are substituted into the index formula, the resulting values should be satisfactorily close to the corresponding present serviceability ratings. There are also advantages if the index formula can be relatively simple in form and if it depends on relatively few pavement characteristics that are readily measured.

Guided by the discussion of the AASHO Rating Panel as well as by results from early rating sessions, the general mathematical form of the present serviceability index was assumed to be

$$PSI = C + (A_1R_1 + A_2R_2 + \ldots) + (B_1D_1 + B_2D_2 + \ldots) \qquad (1)$$

in which R_1, R_2, ... are functions of profile roughness and D_1, D_2, ... are functions of surface deterioration. The coefficients C, A_1, A_2, ..., B_1, B_2, ... may then be determined by a least squares regression analysis. It is expected, of course, that A_1, A_2, ..., B_1, B_2, ... will have negative signs. To perform the analysis, the PSR for the jth of a set of sections is represented by

$$PSR_j = PSI_j + E_j \qquad (2)$$

where E_j is a residual not explained by the functions used in the index. Minimizing the sum of squared residuals for all sections in the analysis leads to a set of simultaneous equations whose solutions are the required coefficients. The respective effect of adding or deleting terms in Eq. 1 will be to decrease or increase the sum of squared residuals. The change in residual sum of squares can be used to deduce the significance of adding or dropping terms from the index formula.

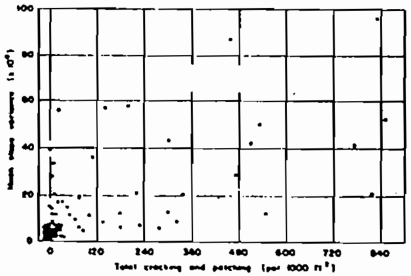

Figure 9. Mean slope variance vs cracking and patching; 74 flexible pavements.

The model for PSI is linear in that if all functions save one are given a numerical value, then PSI versus the remaining function represents a straight line relationship. For this reason it is desirable to choose functions R_1, R_2, ..., D_1, D_2, ..., that have linear graphs when plotted with PSR values. For example, logarithms, powers, etc., of the original measurements may be used as linearizing transformations.

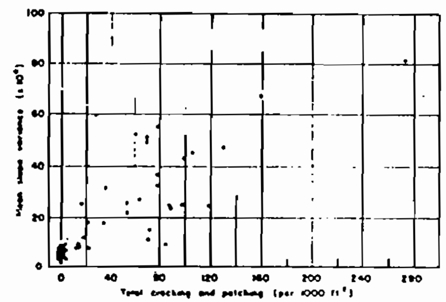

Figure 10. Mean slope variance vs cracking and patching; 49 rigid pavements.

It is important to note that a present serviceability index developed from observed ratings and measurements can only reflect the characteristics that were actually present in the observed pavements. And that for any particular characteristic, the index can only reflect the observed range of values for the characteristic. For example, if the selected pavements had no potholes, there is no objective way to infer how potholing would affect the present serviceability ratings, and the index cannot contain a function of potholing. As another example, if faulting in the selected pavements ranged from 0 to 10, there would be no way to infer the effect on PSR of pavements whose faulting was in the range 50 to 100. This same argument applies to the present serviceability ratings themselves. If PSR's for the selected pavements range only from 2.0 to 4.0, there is no way to infer

what pavement characteristics must be like in order to produce a value of 1.0 or 5.0, say, except to extrapolate the index on the assumption that linearity holds over the full range of pavement characteristics.

For these reasons it has been stated that selected pavements should show all phenomena of interest, the complete range of interest for each phenomenon, and should be associated with PSR values that span the full range of interest.

Thus pavement selection amounts to the assumption that all interesting phenomena and ranges have been encompassed by the selections. Extrapolations of the index to measured values outside the range of those found in the selected pavements amounts to the assumption that the index formula remains linear in the region of extrapolation.

Choice of Functions for the Present Serviceability Index

Measurements from the Illinois and Minnesota sections were plotted in succession against corresponding PSR values to determine which measurements were essentially uncorrelated with PSR and to deduce the need for linearizing transformations. It was indicated that the mean wheelpath slope variance, $\overline{SV}$, was highly correlated with PSR, though curvilinearly. Figures 11 and 12 show the nature of this correlation for all selected pavements. From several alternatives, the transformation

$$R_1 = \log (1 + \overline{SV})$$

was selected as the first function of profile roughness to appear in the PSI model for both flexible and rigid pavements. The result of this transformation is shown in Figures 13 and 14, where PSR values are plotted against R_1 for flexible and rigid pavements, respectively.

For the flexible pavements, mean wheelpath rut depth, $\overline{RD}$, was included as a second profile measurement to appear in the PSI equation. The selected function of rut depth was

$$R_2 = \overline{RD}^2$$

The scatter diagram of PSR vs $\overline{RD}^2$ is shown in Figure 15.

Although preliminary analyses considered the possibility of several functions of surface deterioration (say one function for each of the measured manifestations), it was

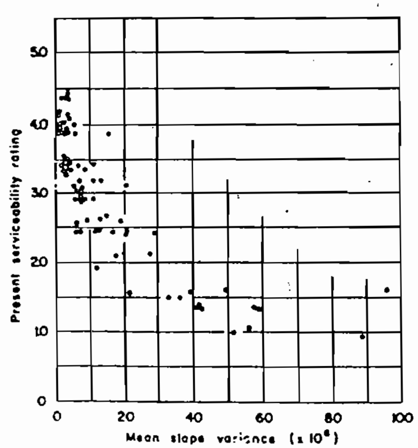

Figure 11. Present serviceability rating vs slope variance; 74 flexible pavements.

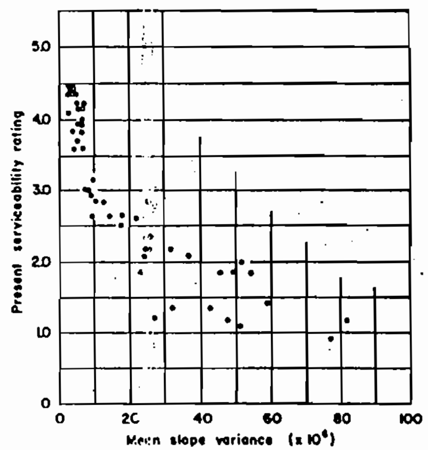

Figure 12. Present serviceability rating vs slope variance; 49 rigid pavements.

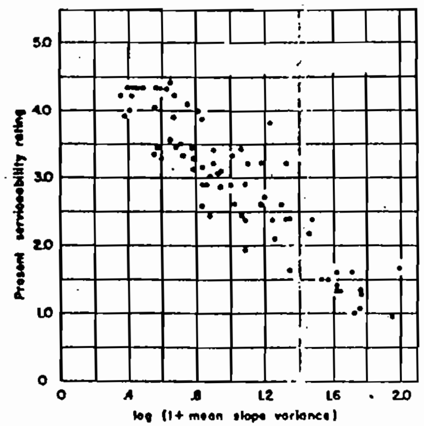

Figure 13. Present serviceability rating vs log (1 + mean slope variance); 74 flexible pavements.

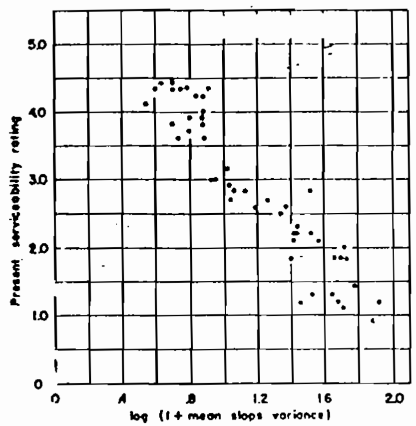

Figure 14. Present serviceability rating vs log (1 + mean slope variance); 49 rigid pavements.

apparent that no loss would be incurred by lumping all major cracking and patching into a single number to represent surface deteriorations. Values for this sum, C + P, are not shown in Tables 1 and 2, but may be obtained from cracking and patching measurements given in the tables.

Scatter diagrams for PSR versus C + P are shown in Figures 16 and 17. For whatever reasons, it is apparent that there is little correlation between PSR and C + P for the flexible pavements, but that a fair degree of correlation exists between these variables for the rigid pavements. For both flexible and rigid pavements the transformation

$$D_1 = \sqrt{C + P}$$

was selected as a linearizing transformation for C + P.

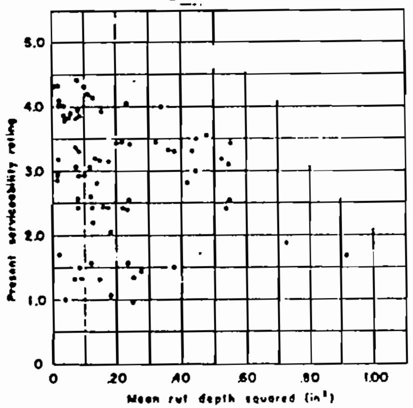

Figure 15. Present serviceability rating vs mean rut depth squared; 74 flexible pavements.

Thus the present serviceability index models to be used are

$$PSI = A_0 + A_1R_1 + A_2R_2 + B_1D_1 = A_0 + A_1 \log (1 + \overline{SV}) + A_2\overline{RD}^2 + B_1 \sqrt{C + P} \qquad (3)$$

for flexible pavements, and

$$PSI = A_0 + A_1R_1 + B_1D_1 = A_0 + A_1 \log (1 + \overline{SV}) + B_1 \sqrt{C + P} \qquad (4)$$

for the rigid pavements. It is not expected that the coefficients A_0, A_1, and B_1 have the same values for both Eqs. 3 and 4.

There are many other possibilities for Eqs. 3 and 4. Not only might other instruments be used to detect deformation and deterioration, but other summary values than $\overline{SV}$, C+P and $\overline{RD}$ might be used. Moreover, one may choose different functions of $\overline{SV}$, C+P and $\overline{RD}$ than appear in Eqs. 3 and 4, or perhaps include still more functions of pavement measurements.

It is clear that one of the most important elements of pavement serviceability is its longitudinal profile in the wheelpaths. The profile of the road coupled with the appropriate characteristics of the vehicle (mass, tires, springs, shock absorbers,

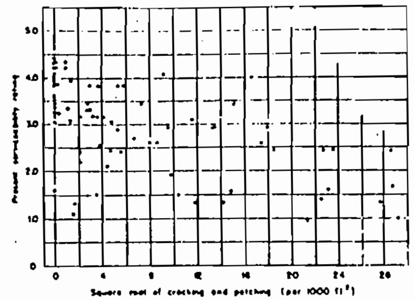

Figure 16. Present serviceability rating vs square root cracking and patching; 74 flexible pavements.

speed, etc.) produce the "ride" attained in that vehicle over that road. The actual profile of the wheel path as though taken with rod and level at very close spacing may be called the displacement profile, p. The first derivative of the displacement profile is the profile of the slope, p'. A plot of the slope profile would have the same abscissa as the displacement profile, distance along the road, and its ordinate would represent the rate of change of displacement, or slope of the road at any point. The second derivative of the displacement profile is the "acceleration" profile, p'', and represents the rate of change of slope, and the third derivative has been called the "jerk" profile, p''', the rate of change of acceleration. It has been suggested that jerk may be more highly correlated with a rider's opinion of his ride than any of the other representations. Perhaps this is true when one is seeking to define "ride" but the efforts at the Road Test were directed toward a definition of the "smoothness of a road" independent of the vehicle that might use it. No small amount of effort was spent in studying correlations of the variances of various profile derivatives with the present serviceability ratings, but there was no evidence that elevation variance, acceleration variance, or jerk variance has higher correlation with PSR than the slope variance. On the other hand, when a number of the slope profiles were subjected to generalized harmonic analysis to determine how variance was associated with the wavelength spectrum, there was some indication that slope variance in certain regions of the wavelength spectrum is more highly correlated with PSR than is the total slope variance. More study of this question is still under way at the Road Test.

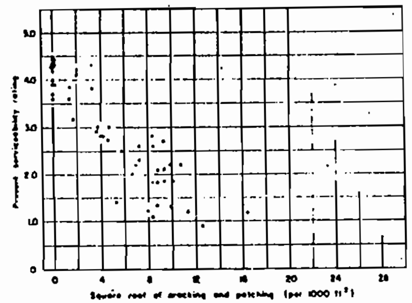

Figure 17. Present serviceability rating vs square root cracking and patching; 49 rigid pavements.

Coefficients for the Present Serviceability Index

Substitution of Eq. 3 in Eq. 2 gives

$$PSR_j = A_0 + A_1R_{1_j} + A_2R_{2_j} + B_1D_{1_j} + E_j$$

in which $R_{1_j} = \log (1 + \overline{SV}_j)$, $R_{2_j} = \overline{RD}_j^2$ and $D_{1_j} = \sqrt{C_j + P_j}$ for the jth pavement.

Least squares estimates for A_0, A_1, A_2 and B_1 are found by minimizing the sum of squared residuals, E_j, through solving the following four simultaneous equations for A_0, A_1, A_2 and B_1.

$$A_1 \sum(R_1-\overline{R}_1)^2 + A_2 \sum(R_1-\overline{R}_1)(R_2-\overline{R}_2) + B_1 \sum(R_1-\overline{R}_1)(D_1-\overline{D}_1) = \sum(R_1-\overline{R}_1)(PSR-\overline{PSR}) \qquad (5a)$$
$$A_1 \sum(R_2-\overline{R}_2)(R_1-\overline{R}_1) + A_2 \sum(R_2-\overline{R}_2) + B_1 \sum(R_2-\overline{R}_2)(D_1-\overline{D}_1) = \sum(R_2-\overline{R}_2)(PSR-\overline{PSR}) \qquad (5b)$$
$$A_1 \sum(D_1-\overline{D}_1)(R_1-\overline{R}_1) + A_2 \sum(D_1-\overline{D}_1)(R_2-\overline{R}_2) + B_1 \sum(D_1-\overline{D}_1)^2 = \sum(D_1-\overline{D}_1)(PSR-\overline{PSR}) \qquad (5c)$$
$$\overline{PSR} = A_0 + A_1\overline{R}_1 + A_2\overline{R}_2 + B_1\overline{D}_1 \qquad (5d)$$

Summations in Eqs. 5 are over all pavements in the analysis, and bars over symbols denote arithmetic means. Sums like $\sum(R_1-\overline{R}_1)^2$ are called sums of squares, while sums like $\sum(R_1-\overline{R}_1)(D_1-\overline{D}_1)$ are called sums of products. Eqs. 5 may be expanded to more terms and more equations if the index model contains more than three functions.

Since the model (Eq. 4) for rigid pavements has only three undetermined coefficients, only three simultaneous equations need be solved.
These equations are

$$A_1 \sum(R_1-\overline{R}_1)^2 + B_1 \sum(R_1-\overline{R}_1)(D_1-\overline{D}_1) = \sum(R_1-\overline{R}_1)(PSR-\overline{PSR}) \qquad (6a)$$
$$A_1 \sum(R_1-\overline{R}_1)(D_1-\overline{D}_1) + B_1 \sum(D_1-\overline{D}_1)^2 = \sum(D_1-\overline{D}_1)(PSR-\overline{PSR}) \qquad (6b)$$
$$\overline{PSR} = A_0 + A_1R_1 + B_1D_1 \qquad (6c)$$

All means, sums of squares, and sums of products for Eqs. 5 and 6 are given in Tables 1 and 2, respectively.

For the flexible pavements, Eqs. 5 are:

$$13.27\ A_1 - 0.166\ A_2 + 171.63\ B_1 = -26.69 \qquad (7a)$$
$$-0.166\ A_1 + 1.34\ A_2 - 3.90\ B_1 = -1.51 \qquad (7b)$$
$$171.638\ A_1 - 3.90\ A_2 + 5255\ B_1 = -369.3 \qquad (7c)$$
$$2.91 = A_0 + 1.02\ A_1 + 0.076\ A_2 + 7.64\ B_1 \qquad (7d)$$

and the solution turns out to give

$$PSI = 5.03 - 1.91 \log(1 + \overline{SV}) - 1.38\ \overline{RD}^2 - 0.01\sqrt{C + P} \qquad (8)$$

For the 49 rigid pavements the least squares equations are:

$$7.55\ A_1 + 71.71\ B_1 = -19.70 \qquad (9a)$$
$$71.71\ A_1 + 905.7\ B_1 = -206.5 \qquad (9b)$$
$$2.83 = A_0 + 1.19\ A_1 - 0.087\ B_1 \qquad (9c)$$

+ 5.19

whose solution leads to the index

$$PSI = 5.41 - 1.78 \log(1 + \overline{SV}) - 0.09\ \sqrt{C + P} \qquad (10)$$

It is noted in Tables 1 and 2 that the total variation in PSR is given by the sums of squares

$$\sum(PSR-\overline{PSR})^2 = 66.85 \text{ for the 74 flexible pavements, and} \qquad (11a)$$
$$\sum(PSR-\overline{PSR})^2 = 57.92 \text{ for the 49 rigid pavements.} \qquad (11b)$$

The variation in PSR as shown by Eqs. 11 may be separated into two parts, a sum of squares attributable to the measured variables and a sum of squares for residuals. Thus,

$$\sum(PSI-\overline{PSR})^2 = \sum(PSI-\overline{PSR})^2 + \sum(PSR-PSI)^2 \qquad (12)$$

when the first term on the right side of Eq. 12 is generally called the sum of squares for regression, or the explained sum of squares. To obtain the sum of squares for regression for the flexible pavements,

$$\sum(PSI-\overline{PSR})^2 = A_1 \sum(R_1-\overline{R}_1)(PSR-\overline{PSR}) + A_2 \sum(R_2-\overline{R}_2)(PSR-\overline{PSR}) + B_1 (D_1-\overline{D}_1)(PSR-\overline{PSR}) \qquad (13)$$

is calculated, then the residual sum of squares is found by subtraction. For the rigid pavements, the term containing A_2 is omitted from Eq. 13. Sums of squares for regression are

$$(-1.91)(-26.69) + (-1.38)(-1.51) + (-0.01)(-369.3) = 56.42 \text{ for}$$

the flexible pavements, and

$$(-1.78)(-19.70) + (-0.037)(-206.5) = 53.08 \text{ for the}$$

rigid pavements.

Dividing regression sums of squares by the total variation given in Eq. 11 gives

$$\frac{56.42}{66.85} = 0.844 \text{ for the flexible pavements, and}$$
$$\frac{53.08}{57.92} = 0.916 \text{ for the rigid pavements.}$$

Thus, the PSI formulas account for 84.4 percent and 91.6 percent of the variation in PSR for flexible and rigid pavements, respectively. By subtractions, the respective sums of squared residuals are 10.43 and 4.84, so that the root mean square residuals are about 0.38 and 0.32, respectively.

The last columns of Tables 1 and 2 show calculated values for the present serviceability indexes, as well as for residuals. At the bottom of the last column of the tables it may be noted that the mean residual was 0.30 for flexible pavements and 0.26 for rigid pavements. In both cases, the mean residual is about twice the mean difference between replicate ratings given by the AASHO Rating Panel.

It may be noted from the residual columns of Tables 1 and 2 that six flexible and three rigid pavement residuals exceeded 0.5, the largest replication difference given by the Panel. However, the index formulas span ratings made more than a year apart, whereas all replicate ratings were made on successive days. As previously stated, it is quite possible that replicate PSR's would be more different when made over larger intervals of time.

When the fifteen rigid pavement PSR values from the fourth rating session were compared with PSI values given by Eq. 10, the sum of the algebraic deviations was practically zero while the mean discrepancy was 0.3. Inasmuch as only two of the deviations exceeded 0.5, it was inferred that Eq. 10 served to fit the new PSR values to about the same degree as it predicted those from which it was derived.

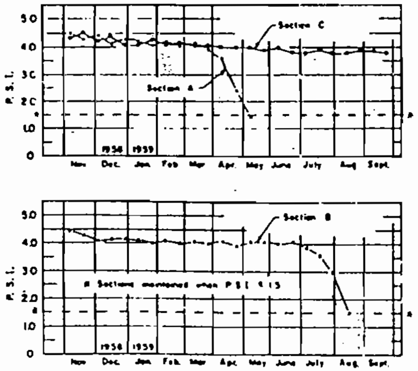

Figure 18. Present serviceability history of three selected test sections on the AASHO road test.

Case Histories of Present Serviceability Index

Figure 18 shows the present serviceability index history of three selected test sections at the AASHO Road Test. Sections A and B have been replaced since the beginning of the test; Section C was still in the test in October 1959. Abscissa values represent two-week intervals for which index values are computed by PSI 111 and PSI 211, respectively.

The performance indexes computed for four dates from these serviceability-time history curves are given in Table 3.

SUMMARY

The fundamental purpose of this paper has been to introduce concepts of present serviceability and performance that can be

TABLE 2

DATA FOR 49 SELECTED RIGID PAVEMENTS

Pvt. Loc.	Sect. Code	Present Serviceability Ratings: AASHO Panel, 1st PSR	AASHO Panel, Replic. diff. in PSR	AASHO Panel, std dev of PSR among raters	Truck Driv'rs PSR	Canad. Raters PSR	Acceptability Opinions, AASHO Panel Fraction: Yes	No	Longitudinal Roughness: $\overline{SV}$ Mean Slope Var'nce in wh'pth (x 10^6)	$\overline{AR}$ Mean AASHO Rbmtr. @ mph (in./mi)	F Faulting in wh'pth in in./1000'	Cracking: C Class 2 and Sealed Cracks ft/1000 ft²	Spalling: ft²/1000ft² for areas > 3" Dia	Patching: P Patch'd Area ft²/1000ft²	Transformations: Log $(1+\overline{SV})$	Log $\overline{AR}$	Sq root C+P	PSI 211 Pres. Serv. Index	Resid. Diff. Betw'n PSR & PSI
Ill.	81	2.0	.2	.6	3.5		.0	.8	52.0		2	53	4	8	1.72		7.8	1.7	.3
	82	4.2		.3	4.5		1.0	.0	6.5		0	4	0	0	.88		2.0	3.7	.5
	83	2.6	.3	.6	2.5		.2	.5	22.2		0	42	0	11	1.77		7.3	2.3	.3
	84	2.3	.2	.3	2.5		.0	.5	36.2		7	16	0	7	1.44		7.3	2.2	.1
	85	1.2		.4	1.5		.0	1.0	47.8		1	132	0	28	1.69		11.4	1.4	.2
	86	2.8	.1	.6	2.5	3.0	.2	.1	25.5		2	15	2	1	1.42		4.0	2.5	.3
	87	4.4	.0	.3	4.5	4.4	1.0	.0	3.2		0	0	0	0	.63		0	4.3	.1
	88	3.1	.2	.4			.0	1.0	50.8		3	65	11	5	1.71		8.4	1.6	.5
	89	0.9	.0	.3			.0	1.0	76.8		1	74	39	85	1.89		12.6	0.9	.0
Minn.	201	1.3	.1	.6			.0	1.0	43.3		1	40	60	99	1.65		10.0	1.6	.3
	202	1.8		.5			.0	1.0	24.2		0	23	4	66	1.60		9.4	2.3	.3
	203	2.1	.3	.6			.1	.9	24.7		0	47	3	43	1.43		9.4	2.1	.0
	204	4.1		.3			1.0	.0	3.4		0	4	0	0	.54		2.0	4.3	.2
	205	3.8	.3	.4			1.0	.0	4.0		0	2	0	0	.70		1.4	4.0	.2
	206	3.0	.0	.5			.6	.2	7.8		1	14	0	1	.95		3.9	3.4	.4
	207	3.0		.6			.4	.2	7.3		1	22	0	0	.93		4.7	3.3	.3
	208	2.9	.1	.6			.3	.4	9.7		0	14	0	0	1.03		3.7	3.2	.3
	209	2.3		.4			.1	.6	17.6		0	34	0	0	1.27		5.8	2.6	.1
	210	1.4		.5			.0	1.0	59.2		0	16	500	12	1.78		5.3	1.8	.4
	211	4.3		.2			1.0	0	3.0		0	0	0	0	.60		0	4.3	.0
	212	4.3	.0	.4			1.0	0	4.0		0	0	0	0	.70		0	4.1	.2
	213	3.7		.4			1.0	0	5.3		0	0	0	0	.80		0	4.0	.3
	214	3.6	.3	.5			1.0	0	4.4		0	0	0	0	.73		0	4.1	.5
	215	3.9		.4			1.0	0	5.3		0	0	0	0	.80		0	4.0	.1
	216	3.9	.0	.6			1.0	0	6.3		0	0	0	0	.87		0	3.8	.1
	217	1.3	.0	.4			.0	1.0	32.3		0	76	2	1	1.52		8.8	1.9	.6
	218	1.2		.4			.0	1.0	27.8		10	64	0	0	1.46		8.0	2.1	.9
	219	2.2		.6			.0	.9	25.6		4	97	0	1	1.42		9.9	2.0	.2
	220	4.4	.0	.3			1.0	.0	4.0		0	0	0	0	.70		0	4.3	.3
Ind.	401	4.0		.3			1.0	0	6.6	134	2	0	1	0	.88	2.13	0	3.8	.2
	402	3.8		.4			1.0	0	6.4	126	4	11	1	0	.88	2.10	3.3	3.5	.3
	403	3.6		.6			.9	0	6.8	133	1	2	4	0	.89	2.04	1.4	3.7	.1
	404	3.2		.6			.6	.2	9.8	131	4	1	1	2	1.03	2.12	1.7	3.4	.2
	405	2.6		.6			.3	.5	14.6	167	5	72	13	0	1.19	2.22	8.5	2.5	.1
	406	2.8		.6			.4	.3	10.4	151	5	70	10	1	1.06	2.18	8.4	2.8	.0
	407	1.8	.5	.6			.1	.8	49.4	268	1	41	4	29	1.70	2.43	8.4	1.6	.2
	408	1.8		.6			.1	.8	54.5	249	2	42	0	37	1.74	2.39	8.9	1.5	.3
	409	2.1		.6			.2	.8	36.6	276	1	50	7	29	1.58	2.44	8.9	1.8	.3
	410	2.2		.5			.2	.8	25.1	230	2	66	5	33	1.42	2.36	10.9	1.9	.3
	411	1.8		.5			.1	.8	45.4	286	0	40	6	65	1.67	2.46	10.2	1.5	.3
	412	2.7		.6			.4	.4	9.9	147	5	81	3	5	1.04	2.17	9.3	2.7	.0
	413	4.2		.4			1.0	.0	6.1	106	1	0	1	0	.85	2.03	0	3.9	.3
	414	4.3		.4			1.0	.0	5.2	112	1	0	0	0	.79	2.05	0	4.0	.3
	415	4.3		.4			1.0	.0	7.1	132	1	0	0	0	.91	2.12	0	3.8	.5
	416	1.2	.3	.6			.0	.9	81.9	338	8	54	1	219	1.92	2.53	14.5	0.5	.7
	417	2.2	.0	.6			.1	.7	32.2	252	10	36	1	0	1.52	2.40	6.0	2.2	.0
	418	4.3	.1	.3			1.0	.0	4.6	113	1	0	0	0	.75	2.06	0	4.3	.2
	419	2.8	.0	.7			.5	.3	12.6	126	2	5	2	13	1.13	2.10	4.2	3.0	.2
	420	2.7	.1	.4			.1	.3	17.8	137	2	5	7	16	1.27	2.14	4.6	2.7	.0
Sum		138.6	3.1												58.23		254.3	138.6*	12.5
Mean		2.83	.13												1.19		5.19	2.83*	.26
Sum of Squares		57.92													7.55		905.70	53.08*	4.84*
Sum of Products with PSR															-19.70		-206.53		
Sum of Products with Log $(1+\overline{SV})$																	71.71		

*Obtained from Unrounded Calculations

$$\text{PSI } 211 = 5.41 - 1.80 \log(1+\overline{SV}) - .09\sqrt{C+P}$$

iceability indexes derived from the first 49 sections. For these reasons, flexible pavements from all four sessions appear in Table 1, but Table 2 includes only rigid pavement sections from the first three sessions.

Present serviceability ratings given in Col. 3, Tables 1 and 2, are mean values for individual ratings given by the Road Test Panel. In general, each mean represents about ten individual ratings. It may be noted that for both pavement types the PSR values range from about 1.0 to 4.5, with nearly the same number of sections in the poor, fair, good, and very good categories. The grand mean PSR for all rated pavements was slightly less than 3.0 for both pavement types.

More than 40 of the sections were revisited by the Panel during the same rating session, and differences between first and second mean ratings are given in Col. 4, Tables 1 and 2. The replication differences ranged from 0 to 0.5, the mean difference being less than 0.2 for both flexible and rigid pavements. Col. 5, Tables 1 and 2, gives the standard deviation of individual PSR values for each section. These standard deviations are of the order 0.5, an indication that only about two or three individual ratings (out of ten) were farther than 0.5 rating points from the Panel mean PSR.

As mentioned earlier, certain of the Illinois sections were rated by two truck drivers, whose mean ratings are given in Col. 6. Col. 7 gives mean ratings given to selected Illinois sections by a group of about 20 Canadian raters. It can be seen that there is general agreement among the various rating groups.

The next two columns of Tables 1 and 2 represent summaries of the AASHO Panel response to the acceptability question. For a particular section the tables show what fraction of the Panel decided the present state of the pavement to be acceptable and what fraction decided the pavement to be unacceptable. By implication the remaining fraction of the Panel gave the undecided response.

Figures 2, 3, 4 and 5 show the connection between corresponding PSR values and acceptability opinions for the two types of pavement. Freehand curves indicate in Figures 2 and 3 that the 50th percentile for acceptability occurs when the PSR is in the neighborhood of 2.9, whereas the 50th percentile for unacceptability corresponds roughly to a PSR of 2.5, as shown in Figures 4 and 5.

Measurements for Selected Pavements

Following the acceptability opinion, Tables 1 and 2 give summary values for measurements made on the selected pavements. Measurements are shown in three categories—those that describe longitudinal and transverse roughness, those that summarize surface cracking and, finally, a measurement of the patched area found in the section.

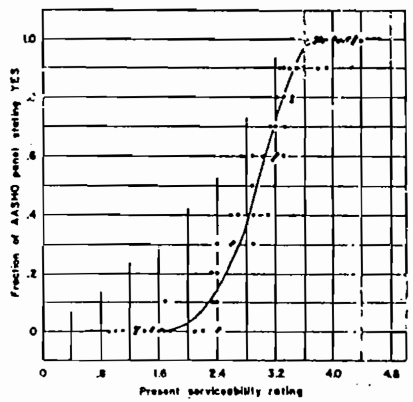

Figure 2. Acceptability vs present serviceability rating; 74 flexible pavements.

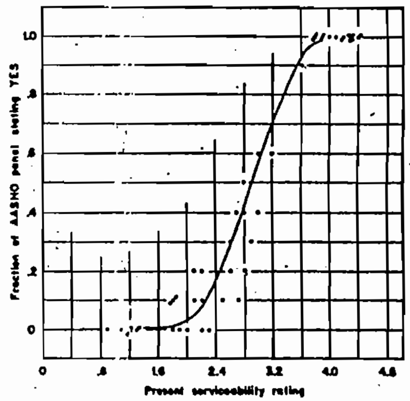

Figure 3. Acceptability vs present serviceability rating; 49 rigid pavements.

Ref: Highway Research Board, "The AASHO Road Test," Report 5, Pavement Research, Special Report 61E, Publication No. 954, National Academy of Sciences - National Research Council, Washington, D.C., 1962.

THE AASHO ROAD TEST
Report 5
Pavement Research

Chapter 1

General Information

1.1 BACKGROUND AND OBJECTIVES

1.1.1 History

The events leading to the three most recent large-scale highway research projects, Road Test 1--MD, the WASHO Road Test and the AASHO Road Test, are described in detail in AASHO Road Test Report 1, "History and Description of the Project" (HRB Special Report 61A). The following is a summary of these events and the activities of the AASHO Road Test.

For many years the member states of the American Association of State Highway Officials had been confronted with the dual problem of constructing pavements to carry a growing traffic load and establishing an equitable policy for vehicle sizes and weights. The Association recognized the common need for factual data for use in resolving the problem. Therefore, in September 1948, it set up a procedure for initiating and administering research projects to be jointly financed by two or more states.

In December of the following year a meeting was held at Columbus, at the request of the Governor of Ohio, to consider the problem of vehicle weight and its effect upon existing and future pavements. The conference was attended by representatives of the Council of State Governments and highway officials of 14 eastern and midwestern states. The need for more factual data concerning the effect of axle loads of various magnitudes on pavements was confirmed.

As a result, Road Test 1–MD was conducted in 1950. An existing concrete pavement in Maryland was tested under repeated application of two single- and two tandem-axle loads. The Highway Research Board administered the test and published the results as HRB Special Report 4.

Concurrently, the Committee on Highway Transport of the American Association of State Highway Officials recommended that additional road tests be initiated by the regional members of the Association. As a result, the Western Association of State Highway Officials sponsored the WASHO Road Test, consisting of a number of specially-built flexible pavements in Idaho tested in 1953-54 under the same loads used in the Maryland test. The results of this test, also conducted by the Highway Research Board, were published as Special Reports 18 and 22.

In March 1951, the Mississippi Valley Conference of State Highway Engineers had started planning a third regional project. However, the idea of another regional project of limited extent was abandoned in favor of a more comprehensive road test to be sponsored by the entire Association. In October, complying with a request by the Association, a Highway Research Board task committee submitted a report, "Proposal for Road Tests," after which the Association appointed a working committee to prepare a prospectus on the project. By December it had been decided to include bridges in the research.

In June 1952, the Working Committee produced a report, "AASHO Road Test Project Statement." In July it selected a site for the project near Ottawa, Ill. In January 1953, it submitted a second report, "AASHO Road Test Project Program," and in August 1954, a third entitled "Project Program Supplement." In May 1955, this committee produced its fourth and final report "Statement of Fundamental Principles, Project Elements and Specific Directions."

Meanwhile, in March 1953, AASHO had formulated a plan for prorating the cost of the project among its member departments and, later, had received assurances of participation from the States, the Automobile Manufacturers Association, the Bureau of Public Roads and the American Petroleum Institute, while the Department of Defense had agreed to furnish military personnel for driving the vehicles.

On February 22, 1955, the Highway Research Board with the approval of its parent organization, the National Academy of Sciences—National Research Council, accepted from the Association the responsibility to administer and direct the new project. The Board opened a field office at Ottawa, Ill., in July 1955; and in August a task force of the Illinois Division of Highways moved to the site to undertake the preparation of plans and to prepare for the construction of the test facilities.

In March 1956, the Board appointed the National Advisory Committee as its senior advisory group and in April selected a project director.

In June 1956, the National Advisory Committee passed a resolution recommending that the Executive Committee of the Highway Research Board consider the inclusion in the facility of a fifth test loop to be subjected to light axle loads. This resolution, recommended by the Bureau of Public Roads, was based on the pending enactment of the Federal Aid Highway Act of 1956. In July, the Executive Committee of the Board approved this change and made additional changes involving special studies areas. The final layout of the test facilities is described in Section 1.2.2.

Construction of the test facilities began in August 1956, and test traffic was inaugurated on October 15, 1958. Test traffic was operated until November 30, 1960, at which time 1,114,000 axle loads had been applied to the pavement and the bridges.

A special studies program was conducted in the spring and early summer of 1961 over some of the remaining test sections. Strains, deflections and pressures were measured in these studies under a wide variety of vehicle types, load suspensions, tires and tire pressures. Special military vehicles, included at the request of the Army, as well as highway construction equipment, were included in these tests. The results of the studies are presented in Road Test Report 6.

During 1961, the research staff concentrated on analysis of the test data and the preparation of reports. Each of the major reports was approved by a review subcommittee of the National Advisory Committee and later submitted to the entire National Advisory Committee and the Regional Advisory Committees prior to its publication by the Highway Research Board. All reports were completed by the project staff, reviewed by the various committees, and submitted to the Board.

The field office for the project was closed in January 1962. However, the Highway Research Board agreed to continue certain studies associated with the Road Test pavement performance analyses in its Washington office. The results of these studies will be reported by the Highway Research Board.

1.1.2 Intent of the AASHO Road Test

The following formal statement of the intent of the Road Test was approved by the Executive Committee of the Highway Research Board January 13, 1961:

> The AASHO Road Test plays a role in the total engineering and economic process of providing highways for the nation. It is important that this role be understood.
>
> The Road Test is composed of separate major experiments, one relating to asphalt concrete pavement, one relating to portland cement concrete pavement, and one to short span bridges. There are numerous secondary experiments. In each of the major experiments, the objective is to relate design to performance under controlled loading conditions.
>
> In the asphalt concrete and portland cement concrete experiments some of the pavement test sections are underdesigned and others overdesigned. Each experiment requires separate analysis. Eventually the collection and analysis of additional engineering and economic data for a local environment are necessary in order to develop final and meaningful relations between pavement types.
>
> All of the short span bridges are underdesigned. Each is a separate case study.
>
> Failures and distress of the pavement test sections and the beams of the short span bridges are important to the success of each of the experiments.
>
> The Highway Research Board of the National Academy of Sciences—National Research Council has the responsibility of administering the project for the sponsor, the American Association of State Highway Officials, within the bounds of the objectives of the test. The Board is also responsible for collecting engineering data, developing methods of analysis and presentation of data, preparing comprehensive reports describing the tests, and drawing valid findings and conclusions. It is here that the role of the Highway Research Board ends.
>
> As the total engineering and economic process of providing highways for the nation is developed, engineering data from the AASHO Road Test and engineering and economic data from many other sources will flow to the sponsor and its member departments. It is here that studies will be made and final conclusions drawn that will be helpful to the executive and legislative branches of our several levels of government and to the highway administrator and engineer.

1.1.3 Objectives

The objectives of the AASHO Road Test as stated by the National Advisory Committee were as follows:

> 1. To determine the significant relationships between the number of repetitions of specified axle loads of different magnitude and arrangement and the performance of different thick-

nesses of uniformly designed and constructed asphaltic concrete, plain portland cement concrete, and reinforced portland cement concrete surfaces on different thicknesses of bases and subbases when on a basement soil of known characteristics.

2. To determine the significant effects of specified vehicle axle loads and gross vehicle loads when applied at known frequency on bridges of known design and characteristics.

3. To make special studies dealing with such subjects as paved shoulders, base types, pavement fatigue, tire size and pressures, and heavy military vehicles, and to correlate the findings of these special studies with the results of the basic research.

4. To provide a record of the type and extent of effort and materials required to keep each of the test sections or portions thereof in a satisfactory condition until discontinued for test purposes.

5. To develop instrumentation, test procedures, data, charts, graphs, and formulas, which will reflect the capabilities of the various test sections; and which will be helpful in future highway design, in the evaluation of the load-carrying capabilities of existing highways and in determining the most promising areas for further highway research.

This report deals primarily with work done in connection with Objectives 1 and 5 and with some of the special studies mentioned in Objective 3. Material relating to Objective 2 will be found in Road Test Report 4 and Objective 4 is discussed in Report 3. Other special studies suggested in Objective 3 are discussed in Report 6.

1.1.4 *Objectivity of Findings*

Discussion of the results given in this report has generally been limited to specific relationships derived from the data. Restraint has been exercised in expressing opinions, conjectures, and speculations. Conclusions have been drawn only when supported by data acquired during the tests.

At the request of the National Academy of Sciences a panel of statisticians was appointed in 1955 so that professional advice was available for both the designs of the Road Test experiments and for the procedures by which the experimental data would be analyzed. It was not the function of this group to select variables nor levels for variables to be included in the Road Test. This was the responsibility of the National Advisory Committee, acting upon the recommendations of the original AASHO Transport Committee's Working Committee. The Statistical Panel played an important role in influencing the experimental layout through its recommendations for complete factorial designs, randomization, and replication. Its recommendations, accepted by the Advisory Committee, made possible effective studies of the relationships sought by the objectives.

Within the space, time and funds available, only a few variables could be studied thoroughly. The experiment was designed and the test facilities built specifically for the study of these variables. In general, mathematical models were used to represent associations among experimental variables, then statistical methods were employed to determine constants for the models as well as to describe the reliability of the evaluated models. Thus experimental designs and analytical procedures were developed in order to obtain unbiased estimates of the effects (and the statistical significance of many of the effects) of controlled experimental factors. The designs and procedures did not, however, make it possible to obtain effects for other factors that were either held constant or that varied in an uncontrolled fashion, for example, embankment soil, strength of materials, and environmental conditions. Although estimates were obtained for the effects of axle load and axle configuration, it was not possible to determine the statistical significance of these effects because replication of load or configuration was not provided. Nevertheless, particularily in the cases of load effect on both pavement types and axle configuration effect on rigid pavement the differences observed were so great as to leave practically no doubt that the effects were significantly greater than zero.

Basic data will be made available to other groups equipped to perform independent analyses. Further analyses are to be encouraged by the Highway Research Board in the expectation that the over-all usefulness of the project will be enhanced.

1.1.5 *Applicability of Findings*

The findings of the AASHO Road Test, as stated in the relationships shown by formulas, graphs, and tables throughout this report, relate specifically to the physical environment of the project, to the materials used in the pavements, to the range of thicknesses and loads and number of load applications included in the experiments, to the construction techniques employed, to the specific times and rates of application of test traffic, and to the climatic cycles that occurred during construction and testing of the experimental pavements. More specific limitations on certain of the findings are given in the discussion of results in various sections of this report. *Generalizations and extrapolations of these findings to conditions other than those that existed at the Road Test should be based upon experimental or other evidence of the effects on pavement performance of variations in climate, soil type, materials, construction practices and traffic.*

1.2 FACILITIES AND OPERATIONS

1.2.1 *Site Location*

The location of the AASHO Road Test was near Ottawa, Ill., in LaSalle County, about 80 mi southwest of Chicago (Fig. 1). The test

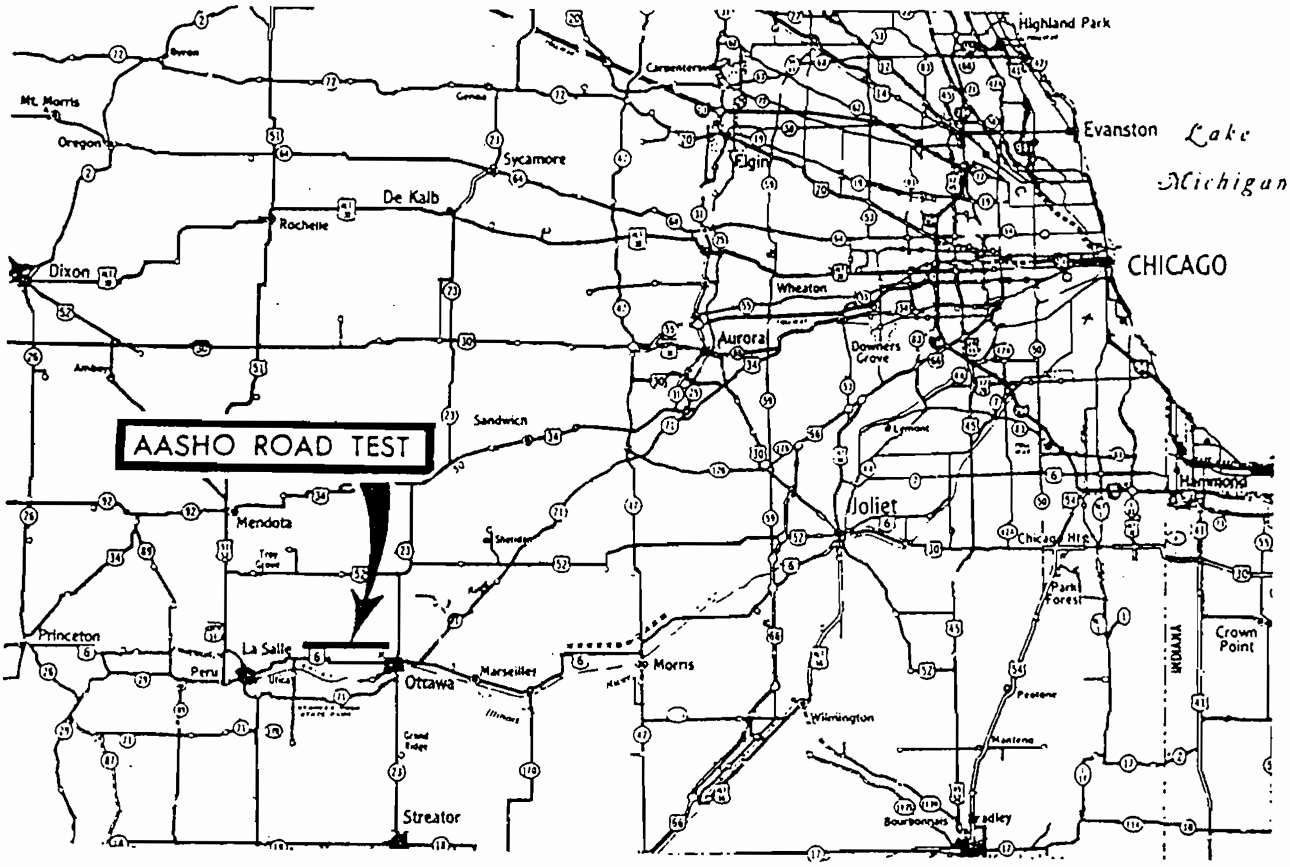

Figure 1. Site location.

facility was constructed along the alignment of Interstate Route 80. The site was chosen because the soil within the area was uniform and of a type representative of that found in large areas of the country, because the climate was typical of that found throughout much of the northern United States, and because much of the earthwork and pavement construction could ultimately be utilized in the construction of a section of the National System of Interstate and Defense Highways.

1.2.2 Test Facilities

The test facilities consisted of four large loops, numbered 3 through 6, and two smaller loops, 1 and 2. Test bridges were at four locations in two of the large loops. The layout of the six test loops, the administration area and the Army barracks is shown in Figure 2.

Each loop was a segment of a four-lane divided highway whose parallel roadways, or tangents, were connected by a turnaround at each end. Tangent lengths were 6,800 ft in Loops 3 through 6, 4,400 ft in Loop 2 and 2,000 ft in Loop 1. Turnarounds in the major loops had 200-ft radii and were superelevated so that the traffic could operate over them at 25 mph with little or no side thrust. Loop 2 had superelevated turnarounds with 42-ft radii. Centerlines divided the pavements into inner and outer lanes, called lane 1 and lane 2 respectively.

All vehicles assigned to any one traffic lane of Loops 2 through 6 had the same axle arrangement-axle load combinations. No traffic operated over Loop 1. In all loops, the north tangents were surfaced with asphaltic concrete and south tangents with portland cement concrete. All variables for pavement studies were concerned with pavement designs and loads within each of the 12 tangents. Each tangent was constructed as a succession of pavement sections called structural sections. Pavement designs, as a rule, varied from section to section. The minimum length of a section was 100 ft in Loops 2 through 6, and 15 ft in Loop 1. Sections were separated by short transition pavements. Each structural section was separated into two pavement test sections by the centerline of the pavement. Figure 3 shows the layout of two typical test loops and locations of the test bridges.

Details of the experiment designs are given in Report 1 and are summarized in Sections 2.1.1 and 3.1.1 of this report. Details concerning all features of bridge research are given in Road Test Report 4.

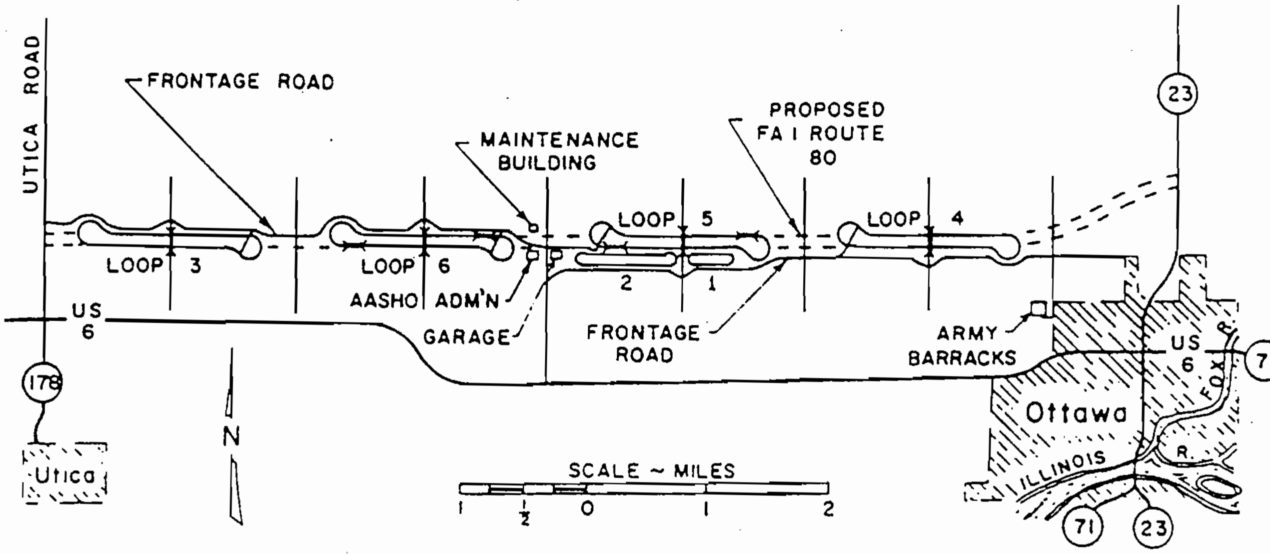

Figure 2. Layout of AASHO Road Test.

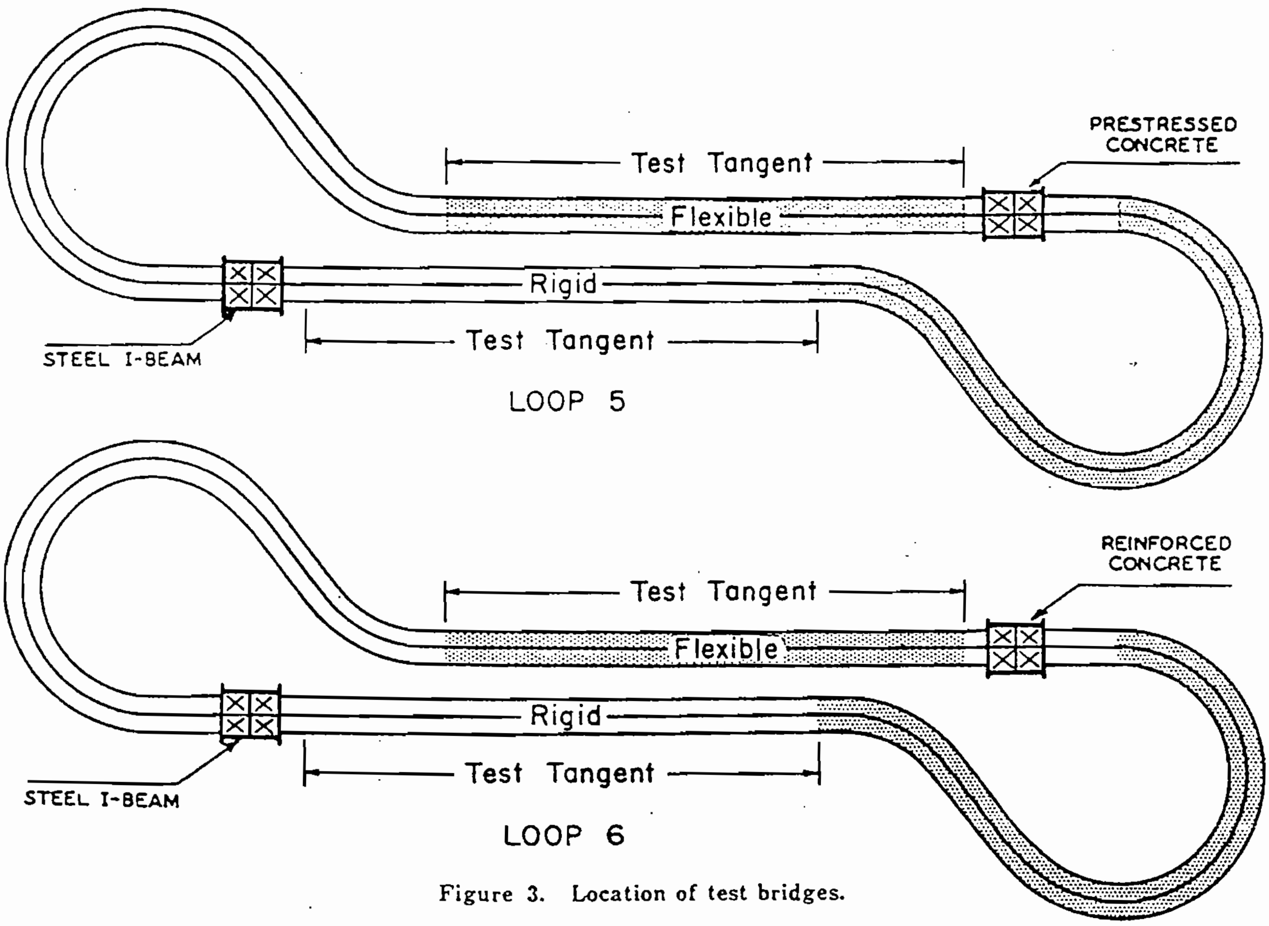

Figure 3. Location of test bridges.

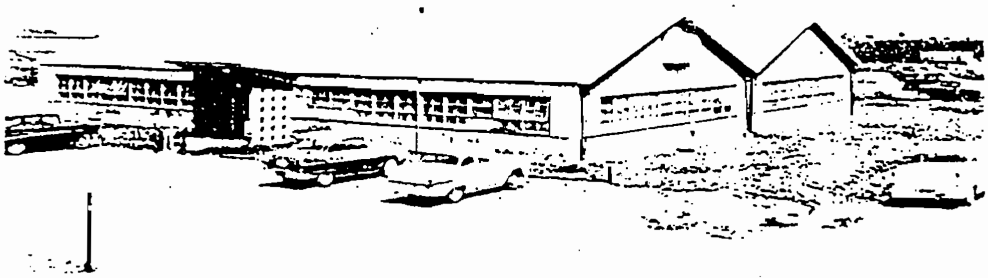

Figure 4. Administration building.

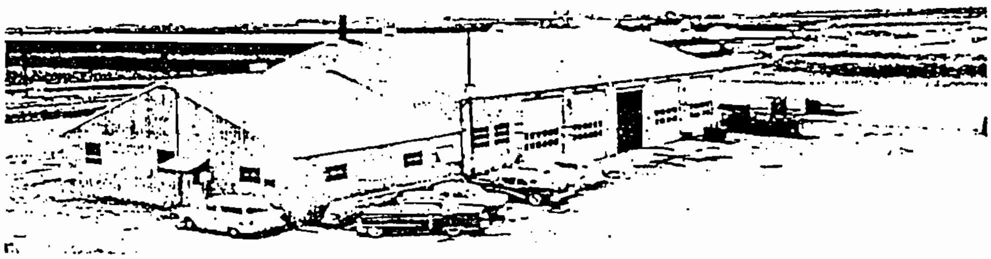

Figure 5. Vehicle maintenance garage.

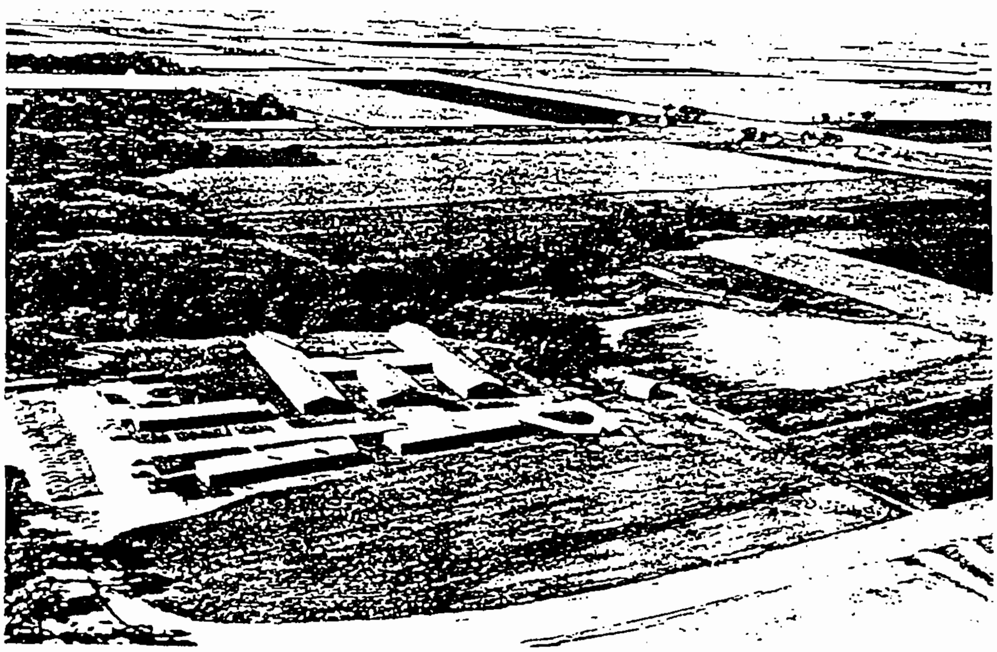

Figure 6. Army driver quarters (Wallace Barracks).

An administrative area was located at the center of the project. Laboratories and offices were located in the building shown in Figure 4. Shop facilities for vehicle maintenance were provided in the building shown in Figure 5. A military installation called Wallace Barracks (Fig. 6) was provided by the National Academy of Sciences to house the Army Transportation Corps Road Test Support Activity.

1.2.3 Construction

A comprehensive description of the construction of the AASHO Road Test facilities is given in Road Test Report 2. Construction was supervised by the task force of the Illinois Division of Highways. On-site materials control and testing were provided by the Highway Research Board Staff on the project. Conventional techniques for construction were used except that extraordinary effort was put forth to insure uniformity of all pavement components. For example, no construction equipment other than that necessary for compaction was permitted to operate in the center 24-ft width of the roadway, and all turning operations on the grade were limited to specially designated transition areas. Specifications for density of compacted embankment soil, subbase and base materials included stipulations of maximum densities as well as the conventional minimums.

Construction was performed under contracts negotiated through normal Illinois contractual channels. It was started in late summer 1956 and completed in time for test traffic to begin in the fall of 1958. S. J. Groves and Sons was the principal contractor in a joint venture with Arcole Midwest, Inc., in the embankment construction and with Rock Roads, Inc., as a subcontractor for asphaltic concrete surfacing. Valley Builders, Inc., built the bridges.

1.2.4 Test Traffic

A detailed description of the operation of the test traffic is presented in Road Test Report 3. As previously stated, Loop 1 was not subjected to test traffic. One lane of this loop was used for subsurface and special load studies, the other for observing the effect of environment on pavements not subjected to traffic. The remaining five loops, 2 through 6, were subjected to traffic for slightly more than two years. Every vehicle in any one of the ten traffic lanes had the same axle load and axle configuration. The assignment of axle loads and vehicle types to the various lanes is shown in Figure 7.

The vehicles were loaded with concrete blocks that were anchored down with steel bands and chains. Although the traffic phase was inaugurated on October 15, 1958, early operation indicated the need to readjust the test loads. This delayed full-scale traffic until November 5, 1958. From November 1958 to January 1960 controlled test traffic consisted of six vehicles in each lane of Loops 3 through 6, four vehicles

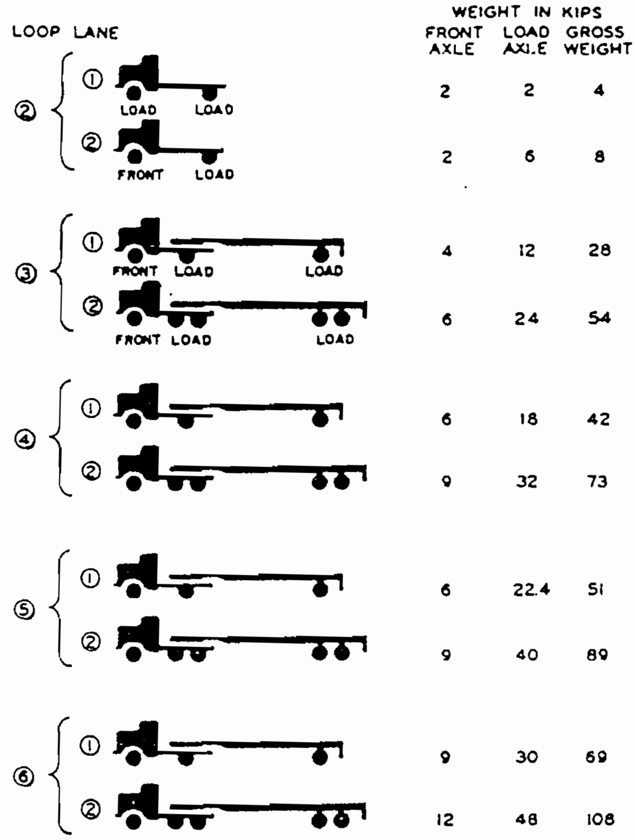

Figure 7. Typical test vehicle axle loadings.

in lane 1 of Loop 2 and eight vehicles in lane 2 of Loop 2. In January 1960, the traffic was increased to ten vehicles in each lane of Loops 3 through 6, six in lane 1 and 12 in lane 2 of Loop 2. These vehicle distributions were selected in order that axle load applications could be accumulated at the same rate in each of the ten traffic lanes.

All lanes had identical specifications for transverse placement, speed, and rate of axle load accumulation. Tire pressure and steering axle loads were representative of normal practice. Some of the vehicles were gasoline and others diesel powered. Further information concerning the vehicles is contained in Road Test Reports 1 and 3.

Whenever possible, traffic was operated at 35 mph on the test tangents. Traffic was scheduled to operate over an 18-hr, 40-min period each day, 6 days a week, except that during the first 6 months of 1960 the schedule was extended to 7 days a week. The schedule was maintained except when pavement distress, truck breakdowns, bad weather and certain other causes made it impossible. A total accumulation of 1,114,000 axle load applications was attained during the 25-month traffic testing period. To accomplish this, soldiers of the U. S. Army Transportation Corps Road Test Support Activity drove more than 17 million miles.

1.2.5 Measurement Programs

Each measurement program was designed to accomplish one or more of the following purposes: (1) to furnish information at regular and frequent intervals concerning the roughness and visible deterioration of the surfacing of each section; (2) to record early in the life of each section transient load effects that might be directly correlated with the ultimate performance of the section; and (3) to furnish a limited amount of additional information which might contribute to a better understanding of pavement mechanics.

Programs falling in the first category were concerned with measurements of permanent changes in the pavement profile along and across the wheelpaths, as well as the extent of cracking and patching of the surfacing. These measurements were given major emphasis since they were used to define the performance of each section as required by the first Road Test objective.

Programs falling in the second category included the measurement of strains and deflections which became the basis for estimating pavement capability, as required by the fifth objective.

Finally, programs of the third category encompassed such measurements as the severity of pumping of rigid pavements, changes in layer thickness in flexible pavements, pavement temperatures, subsurface conditions, and numerous other measurements.

In general, measurements were restricted to those variables that had been demonstrated by previous research to be related significantly to pavement performance. A further restriction, applying especially to subsurface studies, was imposed by the overriding necessity to keep the test traffic moving.

In spite of these restrictions, a formidable amount of data was accumulated, and special electronic systems were evolved to facilitate the storage and initial processing of the data. For example, in the case of some programs, means were provided to record automatically in the field the desired information directly on perforated paper tape, thus eliminating the task of the manual reading of analog records. In another case, an electronic device was used to read field analog records and to punch the information on paper tape for immediate transference to an electronic computer. In general, automatic data handling was used wherever possible and the majority of the data were stored on IBM cards.

Data from the various measurement systems were classified into data systems, and a particular system was identified by a four digit code. Appendix I lists major Road Test data systems concerned with pavement research and notes how the systems may be obtained from the Highway Research Board. Major data systems from the bridge research are listed in Appendix A, Road Test Report 4.

The text of this report contains many references to data systems whose contents are pertinent to the discussion. These references are explained in Appendix I. For example, a reference to Data System 5121, or simply DS 5121, is explained in Appendix I as containing all routine Benkelman beam deflection data for flexible pavement sections on the traffic loops with an IBM printout of the data available on request.

Specific measurement programs are described in the appropriate sections of Parts 2 and 3.

1.2.6 Pavement Maintenance

Detailed descriptions of maintenance criteria and procedures are given in Road Test Report 3. Complete maintenance histories of each test section are available in DS 6300.

The objectives of the Road Test were concerned with the performance of the test sections as constructed. Consequently, maintenance operations were held to a minimum in any section that was still considered under study. When the "present serviceability" (see Section 1.3) of any section dropped to a specified level the section was considered to be out of test and maintenance or reconstruction was performed as needed.

Since the prime objective of the maintenance work was to keep test traffic operating as much as possible, minor repairs were made when required regardless of weather or time of day. The use of pierced steel landing mats permitted traffic to operate through a complete driving period so that more conventional repairs could be made during the daily 5-hr, 20-min traffic break.

All repairs were made with flexible-type pavement material. Deep patches and reconstruction consisted of compacted crushed stone base material surfaced with hot-mixed asphaltic concrete. Overlays consisted of asphaltic concrete. Thin patches were made either with hot-mix or cold-mix materials. Crushed stone base material and cold-mix surfacing were stockpiled at several locations on the project, and hot-mix asphaltic concrete was generally purchased from a nearby contractor.

As a general rule, pavement maintenance was done by project forces with project-owned equipment. However, in the critical spring periods of 1959 and 1960, it was necessary to augment the project maintenance forces with additional men and equipment.

1.2.7 Environmental Conditions

The topography of the Road Test area is level to gently undulating with elevations varying from 605 to 635 ft. Drainage is provided by several small creeks which are tributaries of the Illinois River. Surface drainage, how-

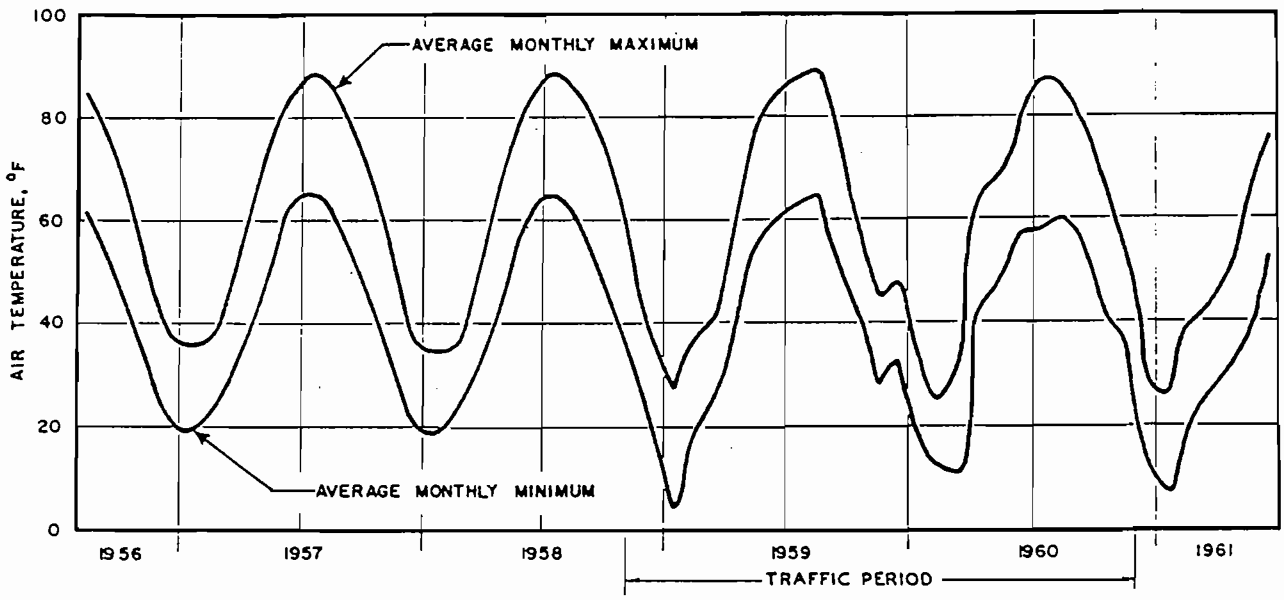

Figure 8. Average monthly air temperature at project.

ever, is generally slow. Geologic information indicates that the area was covered by ice during several glacial periods and that the subsurface soils were deposited or modified during these periods. Surface soils were subsequently derived from a thin mantle of loess deposited during a post-glacial period and were reasonably uniform in the area of the project. Soil drainage is generally poor. Bed rock is found 10 to 30 ft below the surface.

The upper layer of soil was from 1 to 2 ft thick and consisted generally of A–6 or A–7–6 soil with similar characteristics. The adjacent underlying stratum was usually from 1 to 2 ft thick and most of this material was fairly plastic A–7–6 soil. Substratum layers were usually represented by samples exhibiting A–6 characteristics.

In the interest of uniformity, soil making up the top 3 ft of embankment directly under the test pavements was taken from borrow areas near the project. This soil, underlying the surface stratum, was shown by tests to have a plasticity index from 11 to 15, a liquid limit from 27 to 32, and a grain size distribution of 80 to 85 percent finer than the 200 mesh sieve, 58-70 percent finer than 0.02 mm and 34-40 percent finer than 0.005 mm. Maximum dry densities were in the range 114 to 118 lb per cu ft and optimum moisture contents in the range of 14 to 16 percent when compacted in accordance with standard procedure, AASHO T99–49.

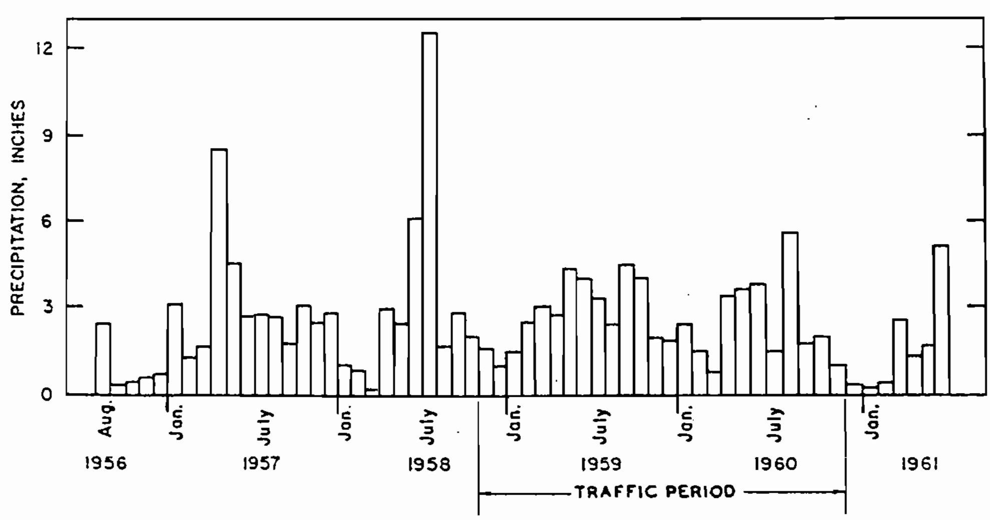

Figure 9. Precipitation at project.

The climate of the Road Test area is temperate with an average annual precipitation of about 34 in. of which about 2.5 in. occurs as 25 in. of snow. The average mean summer temperature is 76 F and the average mean winter temperature is 27 F. The soil usually remains frozen during the winter with alternate thawing and freezing of the immediate surface. Normally the average depth of frost penetration in the area is about 28 in.

Summaries of climatological data observed at weather stations on the project are given in Figures 8 through 10 and frost depth information in Figure 11. Depth of frost under the test pavements was obtained by means of special instrumentation involving the measurement of electrical resistance of the soil as described in *Highway Research Abstracts*, Vol. 27, No. 4. More detailed climatological and frost information is available in the form of IBM listings in Data Systems 3300, 3301, 3140 and 3240. Figure 12 summarizes the observations made at the project on the elevation of the water table under the test pavements and adjacent natural ground.

1.3 PAVEMENT SERVICEABILITY AND PERFORMANCE

1.3.1 Relation to Objectives

The first objective of the Road Test (see Section 1.1.3) asks for relationships between the performance of the pavement and the pavement design variables for various loads. In order to define performance, a new concept was evolved founded on the principle that the prime function of a pavement is to serve the traveling public. Briefly, it was considered that a pavement which maintained a high level of ability to serve traffic over a period of time was superior in performance to one whose riding qualities and general condition deteriorated at a more rapid rate under the same traffic. The term "present serviceability" was adopted to represent the momentary ability of a pavement to serve traffic, and the performance of the pavement was represented by its serviceability history in conjunction with its load application history.

Though the serviceability of a pavement is patently a matter to be determined subjectively, a method for converting it to a quantity based on objective measurements is given in the next two sections. Since the Road Test was concerned only with the structural features of the pavement, such items as grade, alignment, access, condition of shoulders, slipperiness and glare were excluded from consideration in arriving at a value for pavement serviceability.

The serviceability of each test section was determined every two weeks during the traffic testing phase, and performance analyses were based on the trend of serviceability with increasing number of load applications. The serviceability-performance concept is described in detail in Appendix F.

1.3.2 Rating of Pavements in Service

Serviceability was found to be influenced by longitudinal and transverse profile as well as the extent of cracking and patching. The amount of weight to assign to each element in

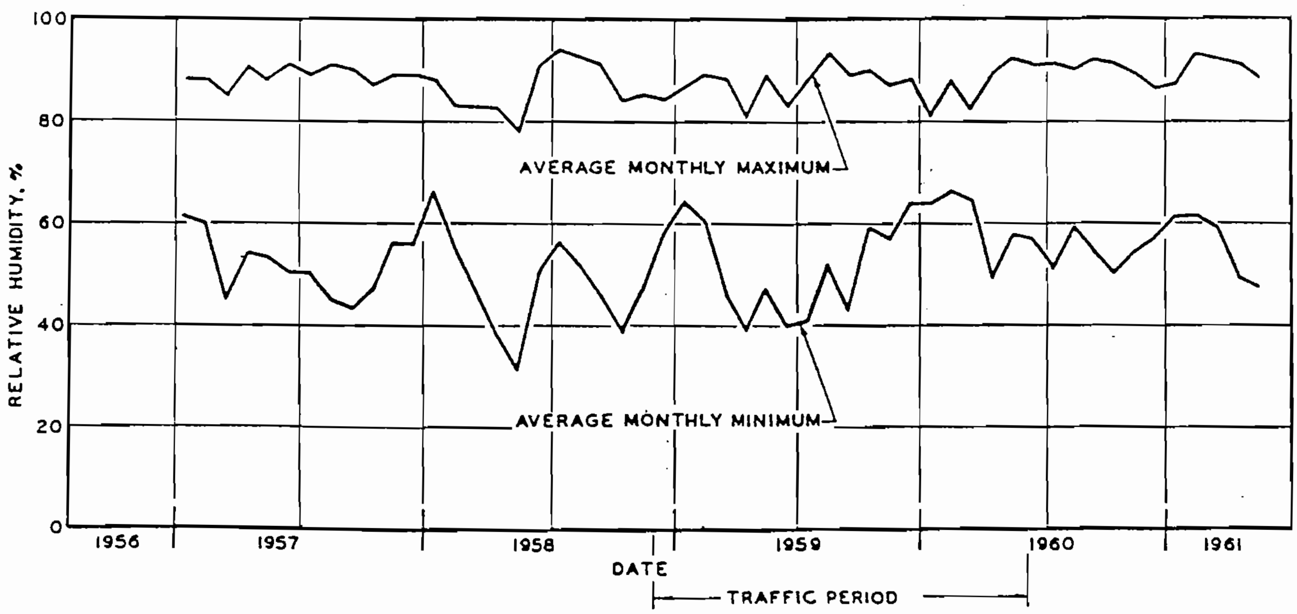

Figure 10. Relative humidity, weather station at Peoria, Ill.

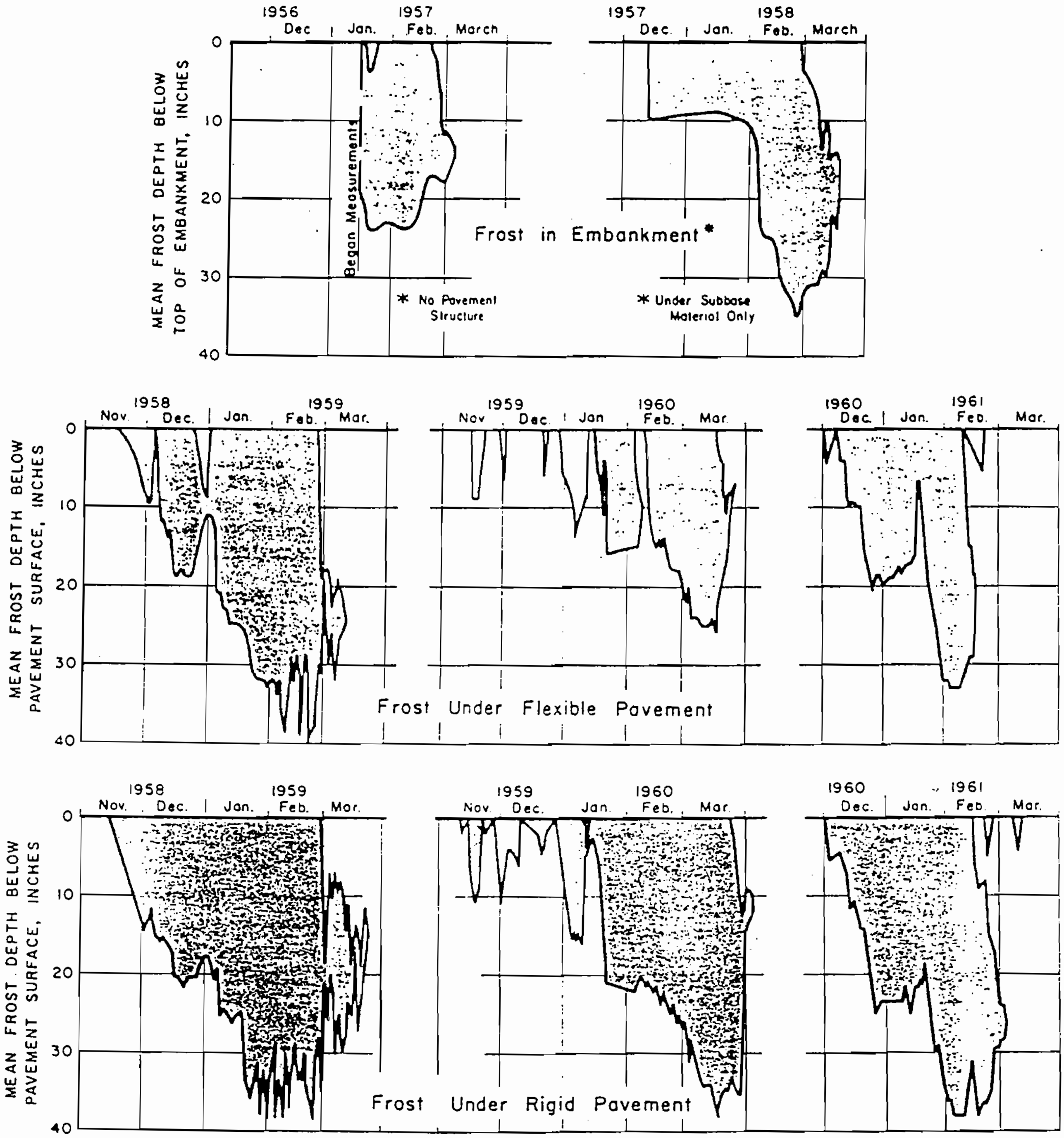

Figure 11. Frost depth.

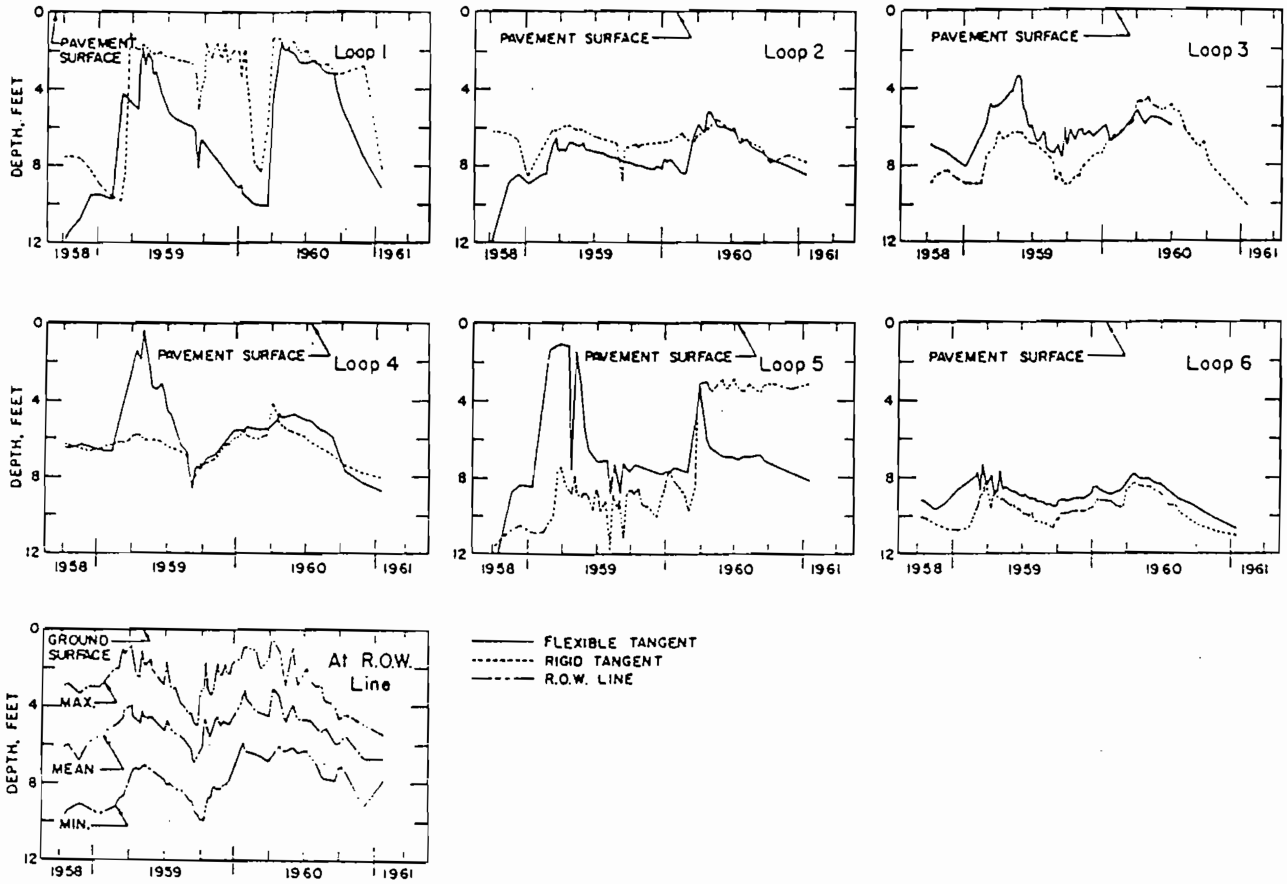

Figure 12. Water table data.

the determination of the over-all serviceability is a matter of subjective opinion. Furthermore, the degree of serviceability loss to be associated with a given change in any one of these elements depends on subjective judgment. To obtain a good estimate of the opinion of the traveling public in these subjective matters a Pavement Serviceability Rating Panel was appointed. This panel included highway designers, highway maintenance men, highway administrators, men with materials interests, trucking interests, automobile manufacturing interests and others. These men made independent ratings of the ability of 138 sections of pavement, located in three states, to serve high speed, mixed truck and passenger traffic. Both rigid and flexible pavements were included, and certain sections were selected for rating in each of five categories ranging from very poor to very good. The members were instructed to use whatever system they wished in rating each pavement and to indicate their opinions of the ability of the pavement to serve traffic at the time of rating on a scale ranging from 0 to 5 with adjective designations of very poor (0–1), poor (1–2), fair (2–3), good (3–4), and very good (4–5). For each section the mean of the independent ratings of the individual panel members was taken as the section's present serviceability rating. Some of the sections were rated more than once in order to determine the ability of the panel to repeat itself. Road Test field crews then measured variations in longitudinal and transverse profiles, as well as the amount of cracking and patching of each section.

1.3.3 Present Serviceability Index

Through a conventional statistical procedure (multiple regression analysis) it was possible to correlate the present serviceability rating with the objective measurements of longitudinal profile variations, the amount of cracking and patching and, in the case of flexible pavements, transverse profile variations (rutting). For either type of pavement this analysis resulted in a formula that used pavement measurements to compute a "present serviceability index" which closely approximated the mean rating of the panel.* The necessary measurements and serviceability index compu-

* A detailed discussion of the work of the Rating Panel, including the ratings, the data obtained in the measurements of the sections that were rated, and the derivation of the present serviceability indexes is presented in Appendix F.

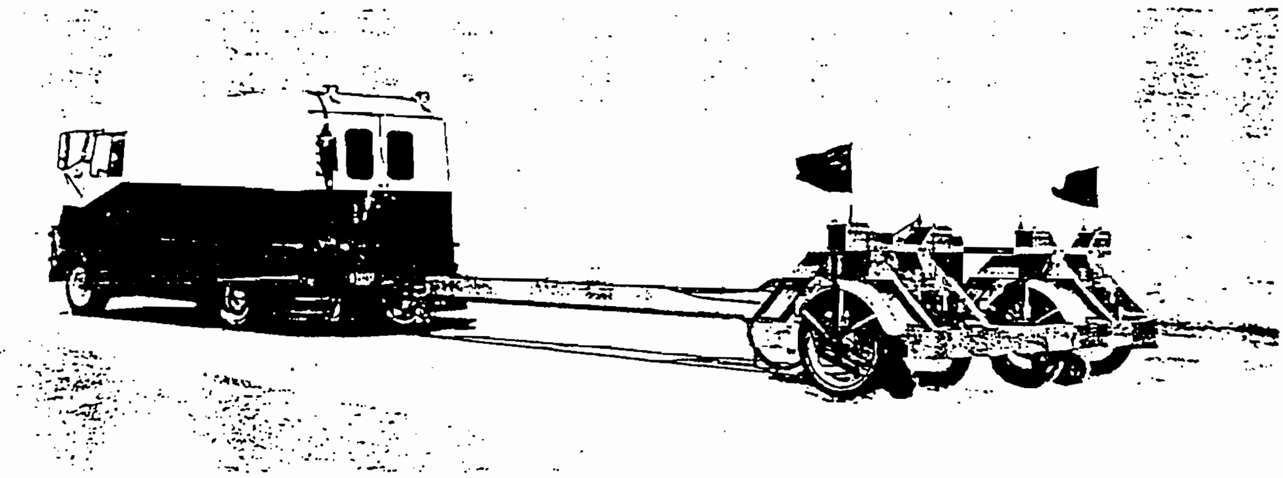

Figure 13. Longitudinal profilometer.

tations were made for each Road Test section at two-week intervals throughout the traffic phase.

Formulas for the present serviceability index, together with descriptions of the measurements entering into them, will be found in Chapters 2 and 3 for flexible and rigid pavement, respectively. The method of measuring longitudinal profile variations was the same for both pavement types and is described below.

The instrument used for recording longitudinal profile variations was the longitudinal profilometer pictured in Figure 13 and shown schematically in Figure 14. This instrument, moving at a speed of 5 mph, recorded continuously the angle, A, formed by the line of the support wheels G and H, and the line CD that connects the centers of two small (8-in. diameter) hard-rubber tired wheels, E, arranged in tandem. One pair of these wheels traveled in the center of each wheelpath.

Since the distance between the centers of the wheels, E, was small (9 in.) the line, CD, was assumed to be approximately parallel to the tangent to the road surface at the point, F, midway between the wheels.

The distance between the supports, G and H, of the tongue being relatively large (25.5 ft), the line GH was regarded as being approximately parallel to the pavement surface had it been perfectly smooth. Thus, the angle, A, between CD and GH represents a departure from a smooth pavement surface and variations in A represent variations in the longitudinal profile. It was this angle that the instrument was designed to measure. The effect of vibration of the tires and springs at G and H was held to a low level by restricting the operating speed and by electrically filtering out high frequencies so that they did not appear on the record.

It was recognized that line GH was not a stable reference and that as a consequence the

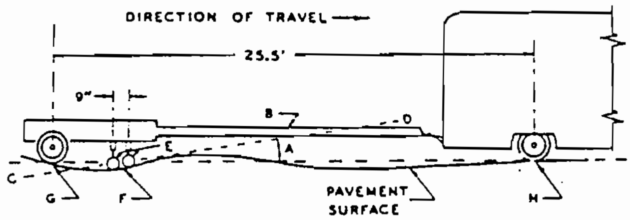

Figure 14. Schematic of longitudinal profilometer.

instrument could not respond correctly to gradual changes in the true pavement slope occurring over relatively long distances. Therefore, considerable effort was expended to develop a means to detect and correct for rotations of the line GH with respect to a horizontal reference. An inertial reference system was devised that would accomplish this purpose for short runs (that is, 2,000 ft). But tests of the effectiveness of the instrument with and without the reference indicated that the inconvenience of operation with the reference far outweighed the small increases in the over-all system effectiveness. Consequently, the inertial reference was abandoned.

The angle A rarely exceeded 3 deg even on rough pavements. Within the range of ±3 deg, the tangent of an angle is virtually equal to the radian measure of the angle, and thus the record of angle A could be interpreted as the slope of the pavement. In this report the profilometer output will be referred to as the pavement slope.

The instrument output on paper tape was a continuous analog of the slope of the pavement in each wheelpath, together with 1-ft distance marks along the margin of the tape (Fig. 15). The tapes were fed into an automatic electronic chart reader (Fig. 16) which measured the ordinate of the chart at intervals equivalent to

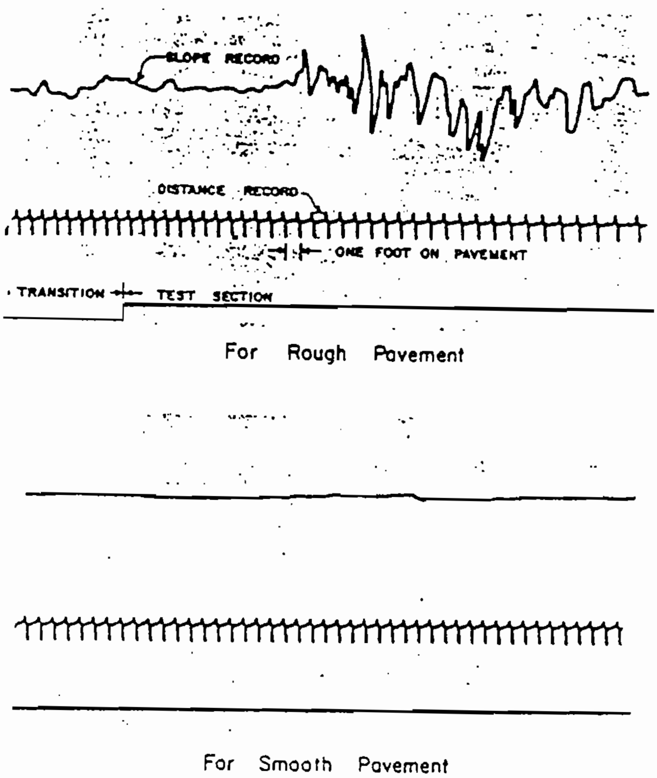

Figure 15. Typical longitudinal profilometer record.

1 ft on the pavement, digitized this information and punched it on perforated paper tape suitable for use as an input to the project's digital computer.

To correlate profile variation with serviceability ratings made by the panel the hundreds of slope measurements taken in each section were reduced to a single statistic intended to represent the roughness of the section. Investigation of several alternative statistics led to the choice of the variance of the slope measurements computed from:

$$SV = \frac{\sum_{i=1}^{n} X_i^2 - \frac{1}{n}\left(\sum_{i=1}^{n} X_i\right)^2}{n-1} \qquad (1)$$

in which

SV = slope variance;
X_i = the i^{th} slope measurement: and
n = total number of measurements.

The slope variance for each section was calculated by the digital computer directly from the tape output of the chart reader. For use by other agencies, the Road Test staff has developed a simplified profilometer (Fig. 17), designated the CHLOE Profilometer, whose

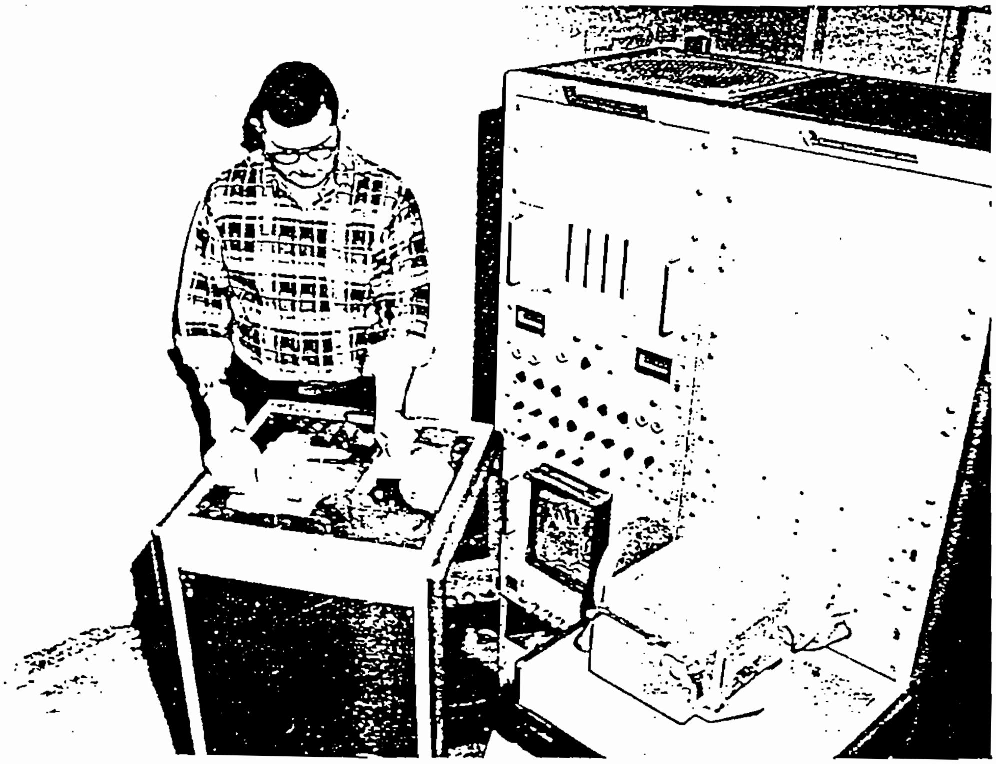

Figure 16. Electronic analog chart reader.

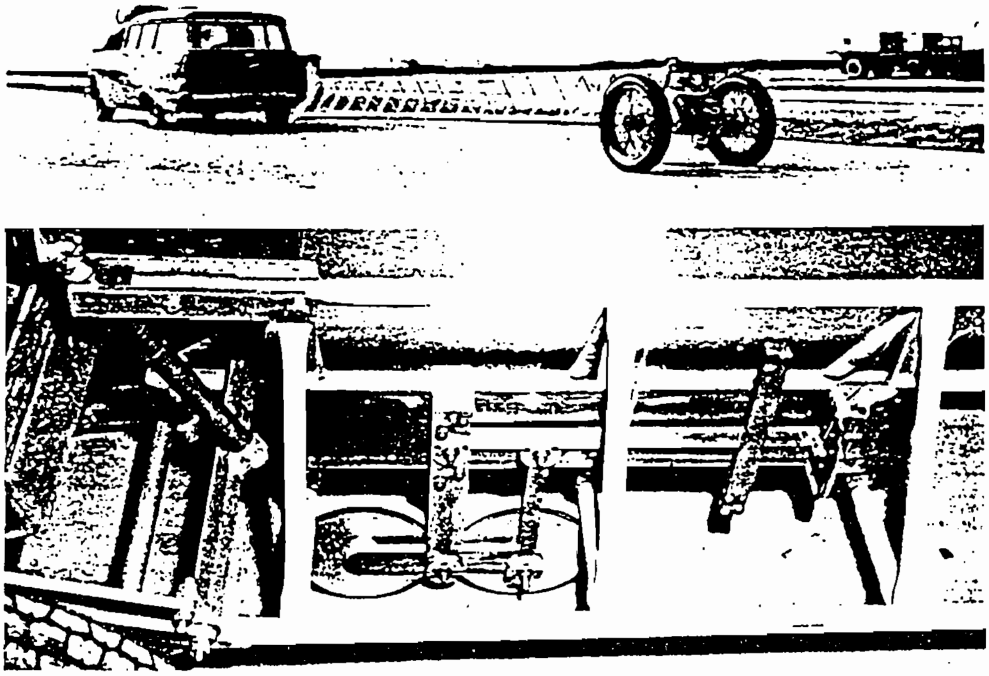

Figure 17. CHLOE profilometer.

output is slope variance. Thus, neither a chart reader nor a digital computer is required when the CHLOE Profilometer is used.

It was found that of the several types of measurements used in the serviceability index formulas, longitudinal profile variation of a section of pavement when represented by the logarithm of the slope variance correlated most highly with the rating of that section by the panel.

1.3.4 Pavement Performance Data

As stated in Section 1.3.1, pavement performance analyses were based on the trend of the serviceability index (determined at intervals of two weeks, or more often when required) with increasing axle applications. Prior to use in the analyses, performance data were identified and processed.

Each 2-week period was termed an "index period", and the last day of each period was called an "index day". Index days were numbered sequentially from 1 to 55, the first occurring on November 3, 1958, and the fifty-fifth on November 30, 1960. Because all sections had been subjected to almost the same number of applications of axle loads on any given date, the pairing of an index value with an index day was equivalent to specifying the serviceability index corresponding to a given number of axle applications. The symbol p_t' was used to represent the serviceability index of any section as determined by measurements made on the t^{th} index day, and the plot of p_t' versus time was termed the "serviceability history" of a section. (Usually the last three days of an index period were required to make the measurements on all sections for determining p_t'.)

The serviceability history of each section was converted to a "smoothed serviceability history" by a moving average that included at least three (generally five) successive index values except that the end values for the history were sometimes taken as end values for the smoothed history. Typical serviceability data and smoothed serviceability histories are shown in Figure 18.

The number of axle applications applied during the t^{th} index period, averaged over the ten traffic lanes, was represented by n_t, and the total number accumulated through that period by N_t; thus,

$$N_t = n_1 + n_2 + \ldots + n_t \qquad (2)$$

It was observed early in the traffic phase of the Road Test, confirming experience elsewhere, that for sections of insufficient design relative to load, the rate at which pavement damage accumulated with applications of load was affected by seasonal changes, especially in the case of flexible pavements. The design of the Road Test experiment did not permit a clearcut comparison of the damage rate in the various seasons since sections which failed in one season were not available for observation during subsequent seasons. Nevertheless Table 1, giving the percentage of failures occurring in each season for each type of pavement, suggests that the damage rate was relatively low in winter for both types of pavement and relatively high in spring for flexible pavements.

Changes in the effect of load with seasons

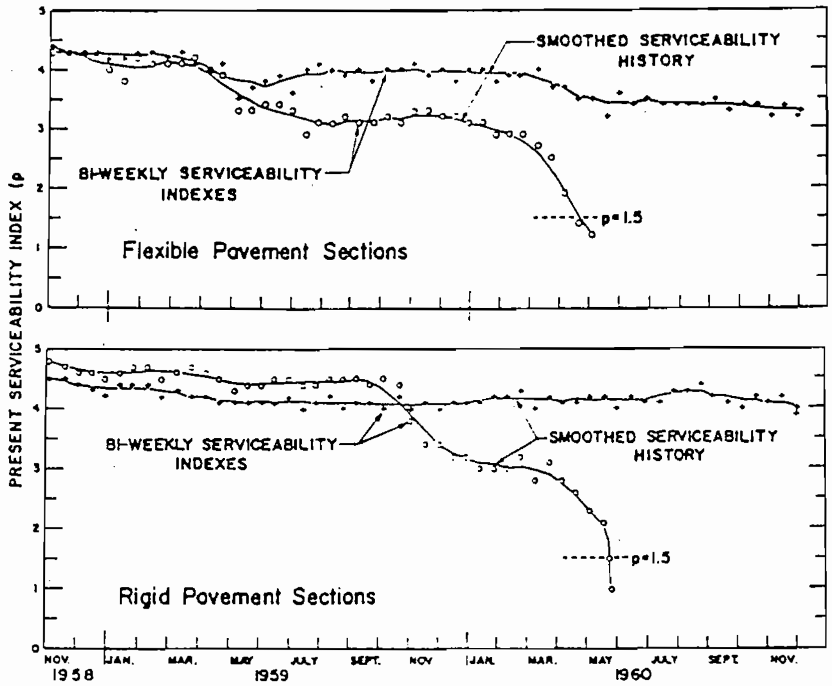

Figure 18. Typical serviceability histories.

TABLE 1
PAVEMENT FAILURE, BY SEASONS

Season	Axle Load Applications ($\times 10^4$)	Seasonal Distribution Section Failure[1] (%) Rigid	Flexible
Fall			
1958 Oct., Nov.	9	0	3
1959 Sept., Oct., Nov.	109	28	1
1960 Sept., Oct., Nov.	173	12	1
All	291	40	5
Winter			
1958-59 Dec., Jan., Feb.	64	0	4
1959-60 Dec., Jan., Feb.	167	11	5
All	231	11	9
Spring			
1959 March, April, May	59	0	57
1960 March, April, May	215	22	23
All	274	22	80
Summer			
1959 June, July, Aug.	109	3	3
1960 June, July, Aug.	209	24	3
All	318	27	6
Total	1,114	100	100

[1] A section was considered to have failed when its serviceability index dropped to 1.5. Table includes only factorial sections (first replicates) in Design 1.

suggested the use of a "seasonal weighting function," q_t, to be multiplied by the number of load applications made during each index period, with the value of q_t depending on some measurement designed to reflect the general variation above and below a "normal" value in the strength of the test sections. The function q_t presumably would take on values greater than unity during periods when the pavement was weaker than normal, and between 0 and 1 when stronger than normal. The product, $q_t n_t$, would then yield "weighted applications," w_t, corresponding to the actual application, n_t, made on each test section during an index period. The total number of weighted applications, W_t, would be given by

$$W_t = q_1 n_1 + q_2 n_2 + \ldots + q_t n_t \qquad (3)$$

Weighted application, W_t, could then be substituted for actual applications, N_t, in the performance analyses. (Hereafter W will be used to represent either weighted or unweighted axle applications, the meaning of the symbol being specified wherever used.)

A seasonal weighting function, dependent on the periodic measurement of flexible pavement deflections in Loop 1, was developed and used in an analysis of flexible pavement performance described in Section 2.2. In the case of rigid pavements, although all rigid pavement distress was associated with pumping and although pumping must be associated with periods of high rainfall, the seasonal variations in damage rate were less pronounced, and no effective function was developed.

For the analyses of pavement performance it was assumed that the trend of serviceability, p, with increasing axle application, W, could be satisfactorily represented by five pairs of coordinates. For sections that failed during the test period, simultaneous values of p and W were taken at $p = 3.5$, 3.0, 2.5, 2.0 and 1.5. For sections that survived the traffic testing period, the coordinates were chosen from the smoothed serviceability history at 11, 22, 33, 44 and 55 index days. Sets of coordinates from the serviceability trend, that is, performance data, for each Road Test section are given in Appendix A.

1.3.5 Procedures for Analysis

The analyses of performance resulted in empirical formulas wherein performance was associated with load and pavement design variables. To use mathematical procedures for the analyses it was necessary to assume some analytical form or model for these associations. In addition to the experimental variables the models include constants whose values were either to be specified or to be estimated from the data. Thus the analytical procedures were for the estimation of constants whose values were unspecified in the model—constants that indicate the effects of design and load variables upon performance. The procedures also included methods for estimating the precision with which the data fit the assumed model. The procedures used in the Road Test analyses are set forth in detail in Appendix G.

There are many different mathematical forms that could be used as models for serviceability trends, and several of these may fit the data with more or less the same precision. Different models were tested for goodness of fit to the Road Test performance data. Preference for one model over another was governed mainly by relative goodness of fit, but consideration was also given to relative agreement with highway design practice and experience for traffic conditions beyond the Road Test.

The mathematical model ultimately chosen for both the flexible and rigid pavement analyses is of the form

$$p = c_o - (c_o - c_1)\left(\frac{W}{\rho}\right)^{\beta} \qquad (4)$$

in which

$c_1 \leqq p \leqq c_o$;

p = the serviceability trend value;

c_o = the initial serviceability trend value (for the Road Test $c_o = 4.5$ for rigid pavements, and 4.2 for flexible pavements—these values were the means of the initial serviceability of test sections);

c_1 = the serviceability level at which a test section was considered out of test and no longer observed (for the Road Test $c_1 = 1.5$);

W = the accumulated axle load applications at the time when p is to be observed and may represent either weighted or unweighted applications.

ρ and β are functions of design and load to be discussed later. Rearranging Eq. 4 in logarithmic form, and defining G, a function of serviceability loss, as $\log (c_o - p)/(c_o - c_1)$ gives

$$G = \beta\ (\log W - \log \rho) \qquad (5)$$

Plotting G against $\log W$ for Eq. 5 gives a straight line whose slope is β and whose intercept on the $\log W$ axis is $\log \rho$. For each Road Test section the performance data given in Appendix A were converted into values for G and $\log W$ and a straight line was fitted to the G, $\log W$ points. From these straight lines, estimates of β and $\log \rho$ were obtained for each test section. For the cases where the serviceability loss was very small over the traffic testing period β may be nearly zero and $\log \rho$ extremely large. Special rules were applied for these cases in order to obtain logical values of β and $\log \rho$ (see Appendix G).

The assumed relationship between β and the design and load variables was

$$\beta = \beta_0 + \frac{B_o\,(L_1 + L_2)^{B_2}}{(a_1D_1 + a_2D_2 + a_3D_3 + a_4)^{B_1}\,L_2^{B_3}} \qquad (6)$$

in which

β_o = a minimum value assigned to β;

L_1 = the nominal load axle weight in kips (*e.g.*, for 18,000-lb single axle load, $L_1 = 18$; for 32,000-lb tandem axle load, $L_1 = 32$);

L_2 = 1 for single axle vehicles, 2 for tandem axle vehicles;

D_1, D_2 and D_3 = the three pavement design factors surfacing, base and subbase thickness for flexible pavement and reinforcement, slab thickness and subbase thickness for rigid pavement.

The remaining symbols of Eq. 6 are positive constants whose values were either to be assigned as was done for β_o or to be estimated by means of the analysis.

Equations in this same form were determined from analysis of the rigid pavement data and the flexible pavement data, respectively.

The analysis rationale assumes that estimates for β from the equation are better than estimates based only on the individual section performance data. Consequently, the values of β estimated from the equation were used in conjunction with the data to obtain new estimates of log ρ for every test section.

The algebraic form assumed for the association of ρ with the design and load variables is

$$\rho = \frac{A_0(D + a_4)^{A_1} L_2^{A_3}}{(L_1 + L_2)^{A_2}} \tag{7}$$

where D $(= a_1D_1 + a_2D_2 + a_3D_3)$ represents a "thickness index" of the pavement, L_1 and L_2 are as defined for Eq. 6, and the remaining symbols are constants whose values are either to be assumed or to be estimated from the analysis.

Evaluation of the constants in Eqs. 6 and 7 is reported in Section 2.2.2 for flexible and 3.2.2 for rigid pavements.

Eqs. 6 and 7 when evaluated and used in conjunction with Eq. 5 thus represent the first goal of the Road Test—to associate performance with design and load variables.

At various stages in the development of the equations, tests were made for the significance of pavement design factors, and statistics were computed to express the degree of correlation between observations and corresponding predictions from the equations. Finally, average residuals were used to indicate the extent to which observations were scattered from the corresponding calculated values of p and log W. Average residuals, correlation indexes, and inferences from the significance tests are summarized after presentation of derived equations in Sections 2.2.2 and 3.2.2.

Many different models and fitting procedures were studied and one selected from which the performance equations fit the Road Test data with satisfactory precision. In time, other models may be found that also fit the data satisfactorily and which may prove equally or more useful.

1.4 NEEDED RESEARCH—GENERAL

1.4.1 Modification of Performance Relationships

Any further effort by the Highway Research Board to fit a mathematical model to the Road Test performance data will likely involve modifications either in the basic models for p, β, and ρ, or in the fitting procedures, or in both. It is the purpose of this section to mention several possibilities for both types of modification that are contemplated in further work with the performance data.

Even if no changes are made in Eq. 4, it is possible to modify the formulas for β and ρ. For example, it might be assumed that β is a constant,

$$\beta = b_0 \tag{8}$$

or that β is a simple function of ρ, for example,

$$\beta = b_0 + \frac{b_1}{\rho\, b_2} \tag{9}$$

The concept of a thickness index for flexible pavements might be generalized after further research to a "structural index," S, where S would account for all pavement layers (their thicknesses and strengths) as well as the embankment soil. A single index for vehicle load, L, might be introduced so that L could account for all axle loads (including steering axles) and their spacing. Then it might be assumed that

$$\rho = \left(\frac{S}{\sqrt{L}}\right)^{A} \tag{10}$$

so that the structural index is squared relative to the load index. It may be noted that the ratio of A_1 to A_2 in Eqs. 18 and 21 (see Section 2.2) is already of the order two to one, so that Eq. 10 appears to be a reasonable assumption at least for flexible pavements.

As is explained in Appendix G, performance equations developed for the present report result from a step-by-step fitting procedure where the results of one step are used as input for the next step. Modification of the fitting procedures will likely take the form of an over-all procedure that determines all unassigned constants simultaneously as a particular residual criterion is minimized. Once the over-all fitting procedure is developed, the residual criterion can include both residuals from log W estimates and residuals from p estimates. Moreover, performance data from experiments that have been analyzed separately in this report may be combined in an effort to obtain a more general analysis.

Although it was not possible to investigate modifications of the type just described in time for inclusion in this report, the Highway Research Board will undertake these studies. It is hoped that further effort will produce modified equations that can represent all the Road Test performance data with at least the same precision as given in this report and that simplifications can be introduced with little sacrifice in precision over the equations reported herein.

1.4.2 Generalization and Extension of Relationships

Discussion in the preceding subsection relates to the need for additional study of the data obtained in the Road Test. A larger area for future research involves the extension of the performance equations to include parameters that were not varied in the project. It is important to know, for example, the effects on pavement performance of variations in the characteristics of the soil and the materials used in the pavement structure. The effects of environment need study. Not only the differences in performance associated with the existence of heavy rainfall, desert conditions, frost, etc., must be considered, but also the differences that may be associated with different rates of traffic application and distribution of axle loads in the traffic stream. (For example, at the Road Test a million axle loads of one weight were applied in two years to each section. What would have been the situation had these loads, accompanied by several million lighter loads, been applied in 20 years?)

Studies designed to fill these gaps may fall in four categories: (1) theoretical studies, (2) major satellite studies, (3) field tests, and (4) laboratory tests.

There should be continuing encouragement of research into the mechanical and physical laws involved in pavement performance. Only through such theoretical work will there be developed rational mathematical models by which performance can be related to the fundamental properties of materials and to the dynamic characteristics of the loading.

Since the completion of such theoretical work appears to be years away, immediate attention should also be given to means for extending the empirical models developed at the Road Test to include additional important parameters. A most effective device for this purpose is the so-called satellite study. These studies have been described* as relatively small road tests in different parts of the country (and other countries) involving consideration of variables most of which were not included in the AASHO Road Test. A very important finding of the Road Test was that, within the range of precision of measurements systems and estimation techniques available, no significant interactions were found among the design variables. Therefore, in the design of satellite experiments where the variables are like those in the Road Test (structure thickness, base type, etc.) balance in the experiment can be attained through the use of partial rather than full factorials.** This means that to test a given number of variables any satellite experiment will require only a small fraction of the test sections that would have been required had the AASHO Road Test shown that significant interactions existed.

Such satellite experiments are also different from the Road Test in that traffic is not a variable. The test sections would be constructed as part of the regular highway system and their serviceability trends observed under the normal traffic using the facility. A careful record of the number and magnitudes of axle loads over the test sections would be required.

These experiments would provide for verification of the coefficients in the Road Test performance equations and for the inclusion of terms in the equations relating to variables that were not under study in the AASHO Road Test. More specific areas for study in the satellite experiments are discussed at the ends of Chapters 2 and 3.

Field tests would be simple pavement performance experiments, with 2 or 3 test sections each, constructed as part of normal highway construction in a large number of locations where only one or two variations from normal pavement design would be observed along with the normal design. These studies would prove very useful to engineers who must use judgment in the application of Road Test findings and in their attempts to evaluate new designs and new materials. However, the field tests would not be designed in such a way as to permit analyses that would result in important modification of the Road Test equations themselves. Many states have constructed test pavements in the field test category in the past. If traffic records are available, further study of these pavements would be extremely useful.

Laboratory tests are those needed in the study of materials characteristics as they might affect pavement performance. Here again more detailed recommendations are given at the ends of Chapters 2 and 3.

1.4.3 Serviceability of Pavements

It is believed that the serviceability-performance concept developed at the Road Test has added a new technique of value in the design and maintenance of highway pavement. It is emphasized, however, that the specific serviceability indexes developed for the Road Test, were based on very small samples of the American highway network by a very small group of highway engineers. There is no reason to think that more extensive sampling will result in major modification of these indexes, but if the system is to receive widespread use, it is imperative that other groups, working under the same rules as the Road Test Rating Panel, make subjective ratings of many sections of pavement over the entire country containing many types of distress leading to loss of serviceability. Accompanying these rating sessions should be objective measurements of those elements that may be involved in serviceability such as, slope variance (roughness), rut depth, cracking, faulting, patching, and slipperiness. Regression analyses of the ratings in terms of the objective measurement data will produce new more generally applicable serviceability indexes.

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* "Extending the Findings of the AASHO Road Test" before the Design Committee, AASHO, at the AASHO meeting in Denver, Colo., October 1961.

** See Hain, R. C., and Irick, P. E., "Fractional Factorial Analysis," HRB Road Test Conference, May 1962.

Ref: Highway Research Board, "The AASHO Road Test," Report 5, Pavement Research, Special Report 61E, Publication No. 954, National Academy of Sciences - National Research Council, Washington, D.C., 1962.

Loop 1

Axle Load: Lane 1 None; Lane 2 None

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
1	0	0	857	858
		8	867	868
		16	833	834
			841	842
	6	0	827	828
		8	847	848
		16	839	840
3	0	0	859	860
			863	864
		8	869	870
			829	830
		16	837	838
	6	0	825	826
			851	852
		8	875	876
			819	820
		16	821	822
5	0	0	823	824
		8	865	866
		16	877	878
	6	0	871	872
		8	849	850
		16	879	880
			873	874

Subsurface Studies — Design 5

Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
3	0	0	861	862
		8	831	832
			853	854
		16	817	818
	6	0	855	856
		8	845	846
		16	843	844
			835	836

Note: Shaded sections are replicates

Loop 2

Axle Load: Lane 1 2,000-S; Lane 2 6,000-S

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
1	0	0	721	722
		4	727	728
	3	0	743	744
		4	717	718
	6	0	755	756
		4	719	720
2	0	0	771	772
		4	729	730
	3	0	759	760
			731	732
		4	741	742
			709	710
	6	0	775	776
			757	758
		4	737	738
			711	712
3	0	0	769	770
		4	739	740
	3	0	773	774
		4	745	746
	6	0	749	750
		4	763	764

Surface Treatment Study — Design 6

Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
Surface Treatment	0	0	733	734
			751	752
		4	753	754
			723	724
	3	0	725	726
			767	768
		4	765	766
			715	716
	6	0	747	748
			735	736
		4	761	762
			713	714

Loop 3

Axle Load: Lane 1 12,000-S; Lane 2 24,000-T

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No. Lane 1	Test Section No. Lane 2
2	0	0	1	165	166
		4	3	125	126
		8	2	143	144
			2	133	134
	3	0	3	113	114
		4	2	135	136
		8	1	159	160
	6	0	2	127	128
		4	1	157	158
		8	3	111	112
3	0	0	2	137	138
		4	1	163	164
		8	3	109	110
	3	0	1	147	148
		4	3	107	108
			3	115	116
		8	2	129	130
	6	0	3	117	118
		4	2	131	132
		8	1	155	156
4	0	0	3	119	120
		4	2	141	142
		8	1	153	154
	3	0	2	145	146
		4	1	151	152
		8	3	121	122
	6	0	1	161	162
			1	149	150
		4	3	123	124
		8	2	139	140

Shoulder Paving Study — Design 2

Shoulder Paving	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
With	2	3	0	177	178
				179	180
	2	3	8	175	176
				183	184
	4	3	0	173	174
				181	182

Base Type Study — Design 4

Base Type	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
Crush. Stone	3	2-14	0	169	170
				105	106

Loop 4

Axle Load: Lane 1 18,000-S; Lane 2 32,000-T

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No. Lane 1	Test Section No. Lane 2
3	0	4	1	633	634
		8	2	607	608
		12	3	571	572
			3	569	570
	3	4	2	599	600
		8	3	573	574
		12	1	617	618
	6	4	3	585	586
		8	1	623	624
		12	2	601	602
4	0	4	3	583	584
		8	1	619	620
		12	2	603	604
	3	4	1	627	628
		8	2	589	590
			2	597	598
		12	3	575	576
	6	4	2	595	596
		8	3	577	578
		12	1	625	626
5	0	4	2	605	606
		8	3	587	588
		12	1	621	622
	3	4	3	579	580
		8	1	631	632
		12	2	593	594
	6	4	1	629	630
			1	615	616
		8	2	591	592
		12	3	581	582

Shoulder Paving Study — Design 2

Shoulder Paving	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
With	3	0	4	637	638
				609	610
	3	6	4	635	636
				611	612
	5	0	4	639	640
				613	614

Base Type Study — Design 4

Base Type	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
Crush. Stone	3	2-16	4	567	568
				561	562

Loop 5

Axle Load: Lane 1 22,400-S; Lane 2 40,000-T

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No. Lane 1	Test Section No. Lane 2
3	3	4	1	485	486
		8	2	451	452
		12	3	415	416
			3	429	430
	6	4	2	449	450
		8	3	419	420
		12	1	487	488
	9	4	3	413	414
		8	1	471	472
		12	2	441	442
4	3	4	3	411	412
		8	1	481	482
		12	2	443	444
	6	4	1	473	474
		8	2	455	456
			2	453	454
		12	3	425	426
	9	4	2	437	438
		8	3	417	418
		12	1	477	478
5	3	4	2	439	440
		8	3	421	422
		12	1	479	480
	6	4	3	423	424
		8	1	469	470
		12	2	445	446
	9	4	1	475	476
			1	483	484
		8	2	447	448
		12	3	427	428

Shoulder Paving Study — Design 2

Shoulder Paving	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
With	3	3	4	435	436
				407	408
	3	9	4	431	432
				405	406
	5	3	4	433	434
				409	410

Base Type Study — Design 4

Base Type	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
Gravel	3	3-18	4	467	468

Loop 6

Axle Load: Lane 1 30,000-S; Lane 2 48,000-T

Main Factorial Design — Design 1

Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No. Lane 1	Test Section No. Lane 2
4	3	8	3	269	270
		12	2	299	300
		16	1	317	318
			1	329	330
	6	8	2	303	304
		12	1	323	324
		16	3	253	254
	9	8	1	321	322
		12	3	267	268
		16	2	309	310
5	3	8	1	319	320
		12	3	261	262
		16	2	315	316
	6	8	3	259	260
		12	2	307	308
			2	305	306
		16	1	327	328
	9	8	2	313	314
		12	1	331	332
		16	3	265	266
6	3	8	2	297	298
		12	1	335	336
		16	3	255	256
	6	8	1	325	326
		12	3	257	258
		16	2	301	302
	9	8	3	263	264
			3	271	272
		12	2	311	312
		16	1	333	334

Shoulder Paving Study — Design 2

Shoulder Paving	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
With	4	3	8	291	292
				275	276
	4	3	16	293	294
				273	274
	6	3	8	295	296
				277	278

Base Type Study — Design 4

Base Type	Surface Thickness	Base Thickness	Subbase Thickness	Test Section No. Lane 1	Test Section No. Lane 2
Crush.	4	3-19	8	287	288

Table 2 Designs for Flexible

section. Figures 19, 20 and 21 are examples of these charts as they may be found for each section in DS 4199.

Basic data relative to the performance of the factorial sections for both weighted and unweighted application are given in Appendix A. Data for a present serviceability level of 1.5 and 2.5, are also given in Tables 5, 6, 7 and 8. Load applications for each design of pavement are given for those sections that were removed from the test and p values for those sections that survived the test.

2.2.2 Performance as a Function of Design and Load

This subsection gives relationships between flexible pavement performance and variables that describe load and pavement design. Performance data, models, and analytical procedures described in Section 1.3 are used to obtain specific performance-design-load equations for the factorial experiments. This section also includes associations of performance with design and load variables for the paved shoulder studies and for the special base type studies.

2.2.2.1 Main Factorial Experiments (Design 1).—This subsection contains the results of the major Road Test flexible pavement analysis, the pavement performance analysis, and develops the relationships for flexible pavement sought in the first objective. These relationships have been reduced to four equations containing terms for the variables included in the test. Eqs. 13, 17, 18, and 19 are for the case where load applications have been adjusted by the seasonal weighting function; similar equations are given for unweighted applications.

Graphs and tables were constructed from the equations for use in the study of performance over the wide range of designs and loads included in the Road Test.

A convenient presentation of the relationships for the axle loadings of the Road Test is shown in Figure 22. For example, to determine what pavement structure would have survived a million 22.4-kip single axle loads at the Road Test before its serviceability level dropped to 2.5, the chart is entered at 1,000,000 applica-

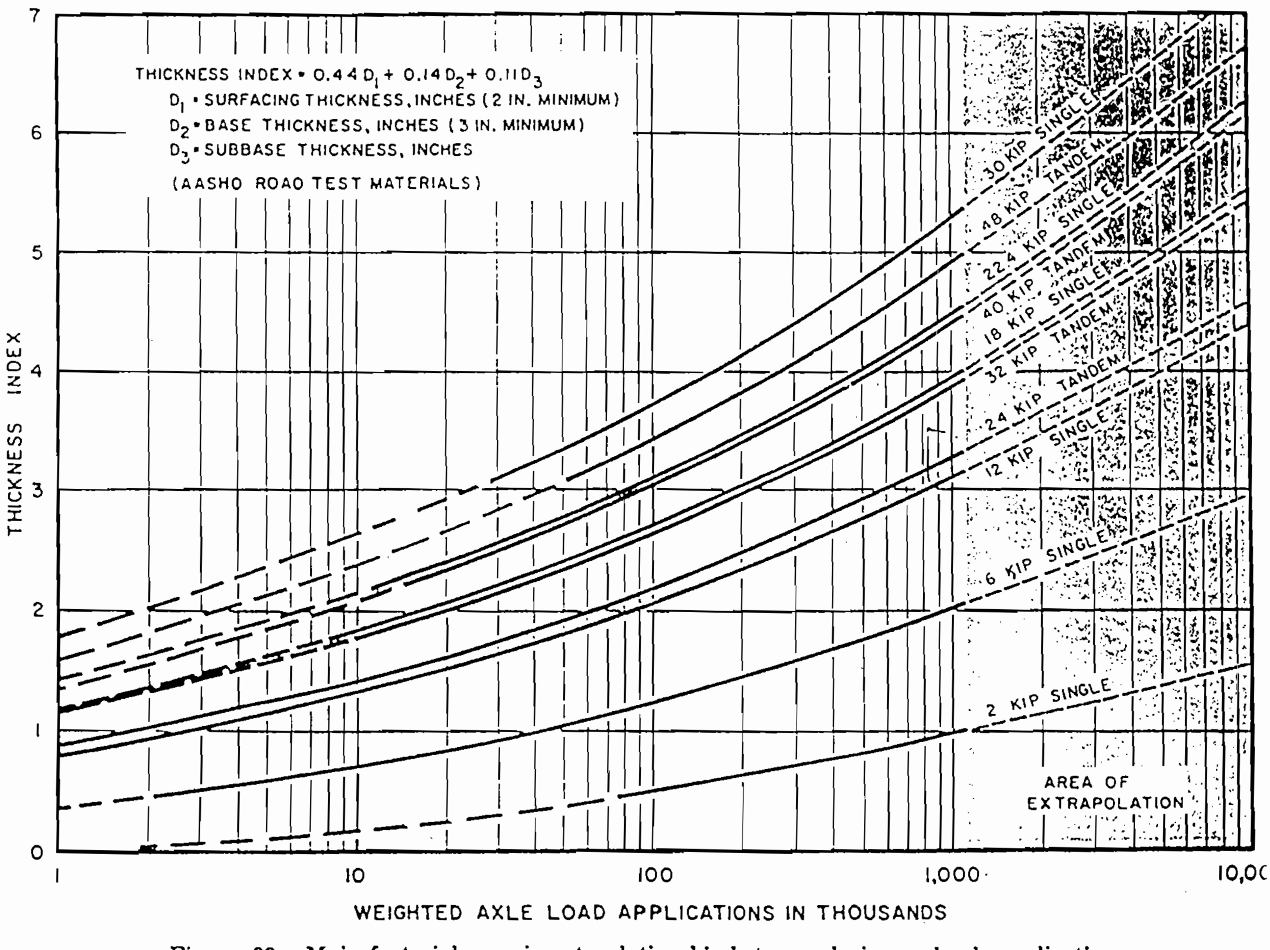

Figure 22. Main factorial experiment, relationship between design and axle application at $p = 2.5$ (from Road Test equations).

tions on the abscissa and the thickness index (4.5) is read on the ordinate scale. Asphaltic concrete surfacing, base and subbase may be combined in any combination for an index of 4.5, provided it meets the conditions for use of the thickness index equation stated on the chart. Many combinations of structural layers will meet these conditions. One, for example, is 4 in. of surfacing, 10 in. of base and 12 in. of subbase.

Since these equations represent serviceability trend data observed in the test, some Road Test sections failed sooner and some later than indicated by the smooth curves. Thus, some allowance should be made for the scatter of the data as shown, for example, in Figure 25. Through a residual analysis it was found that the scatter corresponds to approximately ±14 percent of the thickness index values given by the curves. If comparisons are made with observed performance of actual highways in service, additional allowance should be made to account for differences between the Road Test and the actual highway in materials, environment, and loading history.

These relationships are not intended to be design equations. However, they can serve as a basis for design procedures in which variables not included in the Road Test, such as soil type, are considered.

Tables and discussion are included to show the basis for determining the significance or nonsignificance of the various effects. Correlation indexes show the degree of correlation found in the relationships; mean residuals, the degree of scatter of the observed performance data from the predictions of the performance equations.

The thickness index found to apply to Road Test flexible pavements is of interest in itself. For the weighted applications case the thickness index equation (Eq. 19) indicates that an inch of surfacing was about three times as effective as an inch of base and four times as effective as an inch of subbase in improving pavement performance within the range of design studied.

The use of the seasonal weighting function on axle load applications was found to increase the correlation index from 0.48 to 0.70 and to reduce the mean residuals by 15 percent.

The general model used to represent pavement performance was Eq. 4. For flexible pavement test sections in the factorial experiments the average initial serviceability trend value was $c_0 = 4.2$, and since $c_1 = 1.5$, $c_0 - c_1 = 2.7$, and the trend curves are represented by

$$p = 4.2 - 2.7 \left(\frac{W}{\rho}\right)^{\beta} \qquad (12)$$

Both β and ρ are positive functions of the design variables, D_1 (surfacing thickness, in.), D_2 (base thickness, in.), and D_3 (subbase thickness, in.), and of the load variables, L_1 (nominal axle load, kips*) and L_2 (1 for single axles or 2 for tamdem axles).

The function β determines the general shape of the serviceability trend with increasing axle load applications, W. If $\beta = 1$, the trend is a straight line; if $\beta > 1$, the serviceability loss rate increases with applications; and if $\beta < 1$, the loss rate decreases with axle load repetitions. Graphs of the performance data for flexible pavements in Appendix A indicated that designs failing early in the Road Test tended to have an increasing rate of serviceability loss ($\beta > 1$), while more adequate designs as a rule had a decreasing loss rate ($\beta < 1$). Estimates of β were obtained from the performance data of a number of sections that experienced relatively little serviceability loss in the Road Test. The average of these values was approximately 0.4, and this value was assigned to β_0, the assumed minimum value for β in Eq. 6.

The function ρ is equal to the number of load applications at which $p = 1.5$, and is assumed to increase as design increases and to decrease as load increases. The over-all aim of the performance analysis is to arrive at formulas for β and ρ in terms of D_1, D_2, D_3, L_1 and L_2 so that Eq. 12 may be used to predict the value of p after a specified number of applications, W. Or if Eq. 12 is solved for log W,

$$\log W = \log \rho + \frac{\log\left(\dfrac{4.2 - p}{2.7}\right)}{\beta} \qquad (13)$$

then Eq. 13 may be used to predict the number of applications required to reduce p to a specified value.

For the flexible pavements, β and ρ are given by particular cases of Eqs. 6 and 7 of Section 1.3.5, as follows:

$$\beta = 0.4 + \frac{B_0(L_1 + L_2)^{B_1}}{(D + 1)^{B_2} L_2^{B_3}} \qquad (14)$$

$$\rho = \frac{A_0(D + 1)^{A_1} L_2^{A_3}}{(L_1 + L_2)^{A_2}} \qquad (15)$$

in which D is a thickness index given by

$$D = a_1D_1 + a_2D_2 + a_3D_3 \qquad (16)$$

If the coefficients a_1, a_2 and a_3 in Eq. 16 are each assigned a value of one, D is the total structure thickness. In the Road Test analyses,

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* For example, for single axle loads of 18 or 22.4 kips, L_1 = 18 or 22.4; for tandem axle loads of 32 or 40 kips, L_1 = 32 or 40.

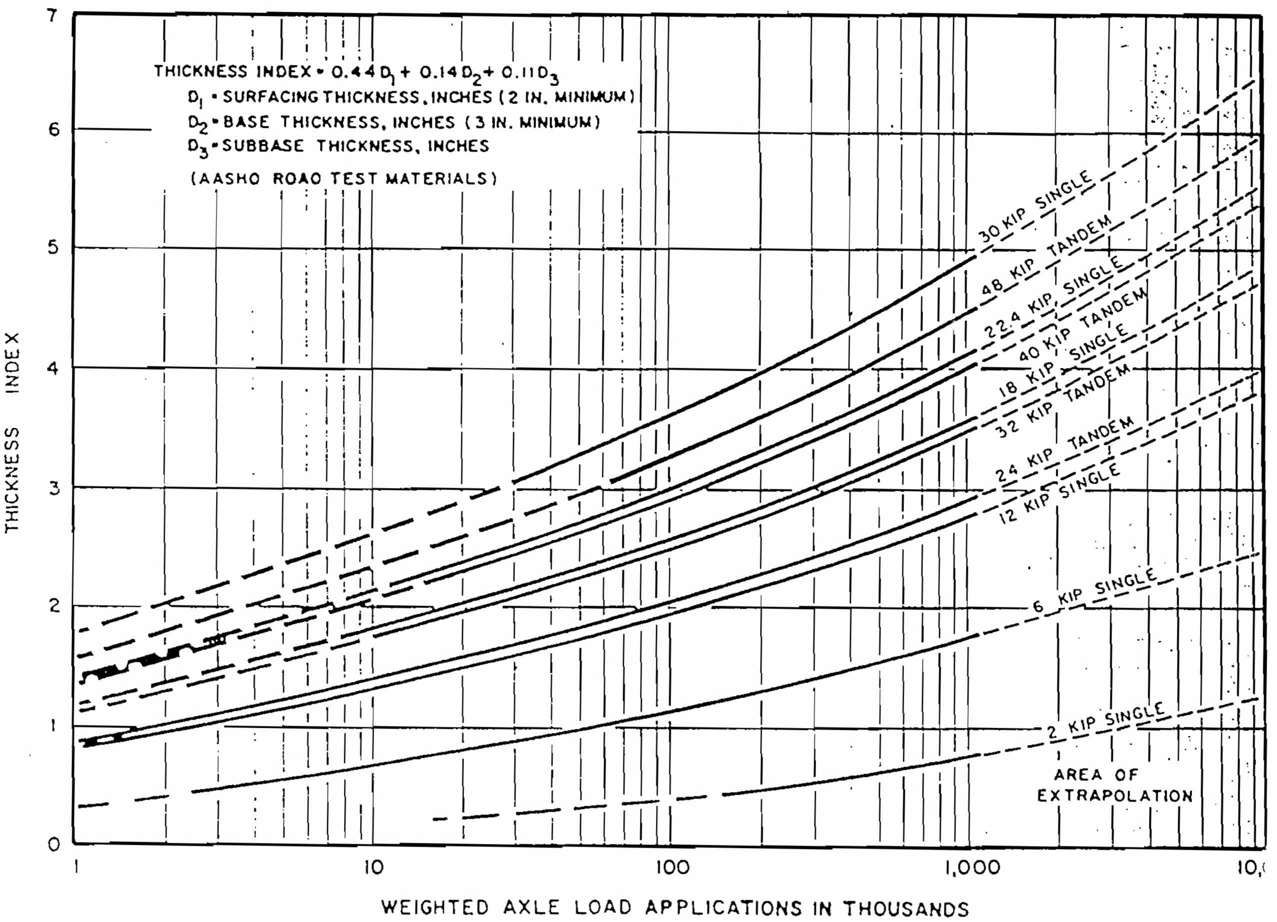

Figure 23. Main factorial experiment, relationship between design and axle load applications at $p = 1.5$ (from Road Test equations).

significant effects. Thus this and similar analyses of variance all pointed to the use of a thickness index as given by Eq. 16.

The third part of Tables 9 and 10 shows within loop estimates for a_1, a_2, and a_3 that were obtained from the variance analyses. Weighted averages of these estimates gave the values shown in Eqs. 19 and 22. The last part shows the results of within lane regression analyses that were used to determine values for A_1 in Eq. 15. In the logarithmic form, A_1 is the coefficient of log $(a_1D_1 + a_2D_2 + a_3D_3 + 1)$, and estimates for this coefficient are shown for each lane at the bottom of the table. Weighted average values for A_1 are 9.36 and 8.94 for the two cases represented by Eqs. 18 and 21. The remaining constants in Eqs. 14 and 15 were determined by applying procedures described in Appendix G to the performance data of Appendix A.

If W represents weighted applications obtained through the use of seasonal weighting

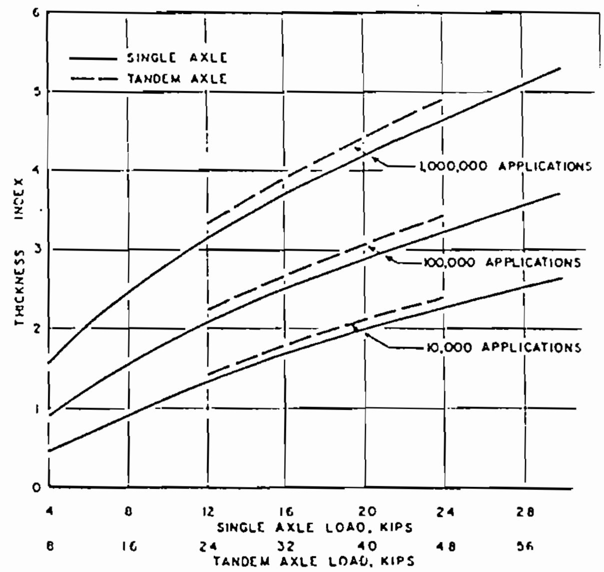

Figure 24. Main factorial experiment, relationsh between design and load at $p = 2.5$.

function described in Section 2.2.2.1.1, then the analysis gives the following equations:

$$\beta = 0.4 + \frac{0.081\,(L_1 + L_2)^{3.23}}{(D + 1)^{5.19}\, L_2^{3.23}} \qquad (17)$$

$$\rho = \frac{10^{5.93}\,(D + 1)^{9.36}\, L_2^{4.33}}{(L_1 + L_2)^{4.79}} \qquad (18)$$

$$D = 0.44D_1 + 0.14D_2 + 0.11D_3 \qquad (19)$$

If the applications are unweighted, then the performance equations are as follows:

$$\beta = 0.4 + \frac{0.083\,(L_1 + L_2)^{4.87}}{(D + 1)^{8.73}\, L_2^{4.87}} \qquad (20)$$

$$\rho = \frac{10^{6.16}\,(D + 1)^{8.04}\, L_2^{4.17}}{(L_1 + L_2)^{4.54}} \qquad (21)$$

$$D = 0.37D_1 + 0.14D_2 + 0.10D_3 \qquad (22)$$

Thus for a particular pavement design and axle load, either Eqs. 17, 18 and 19 or Eqs. 20, 21 and 22 give values for β and ρ that may be substituted in Eq. 12 if p is to be estimated from W, or in Eq. 13 if W is to be estimated when p is given. Figures 22 and 23 show how W varies with D in Eq. 13 when p is fixed at 2.5 and 1.5, respectively. Each figure has ten curves, one curve for each test load used in the Road Test.

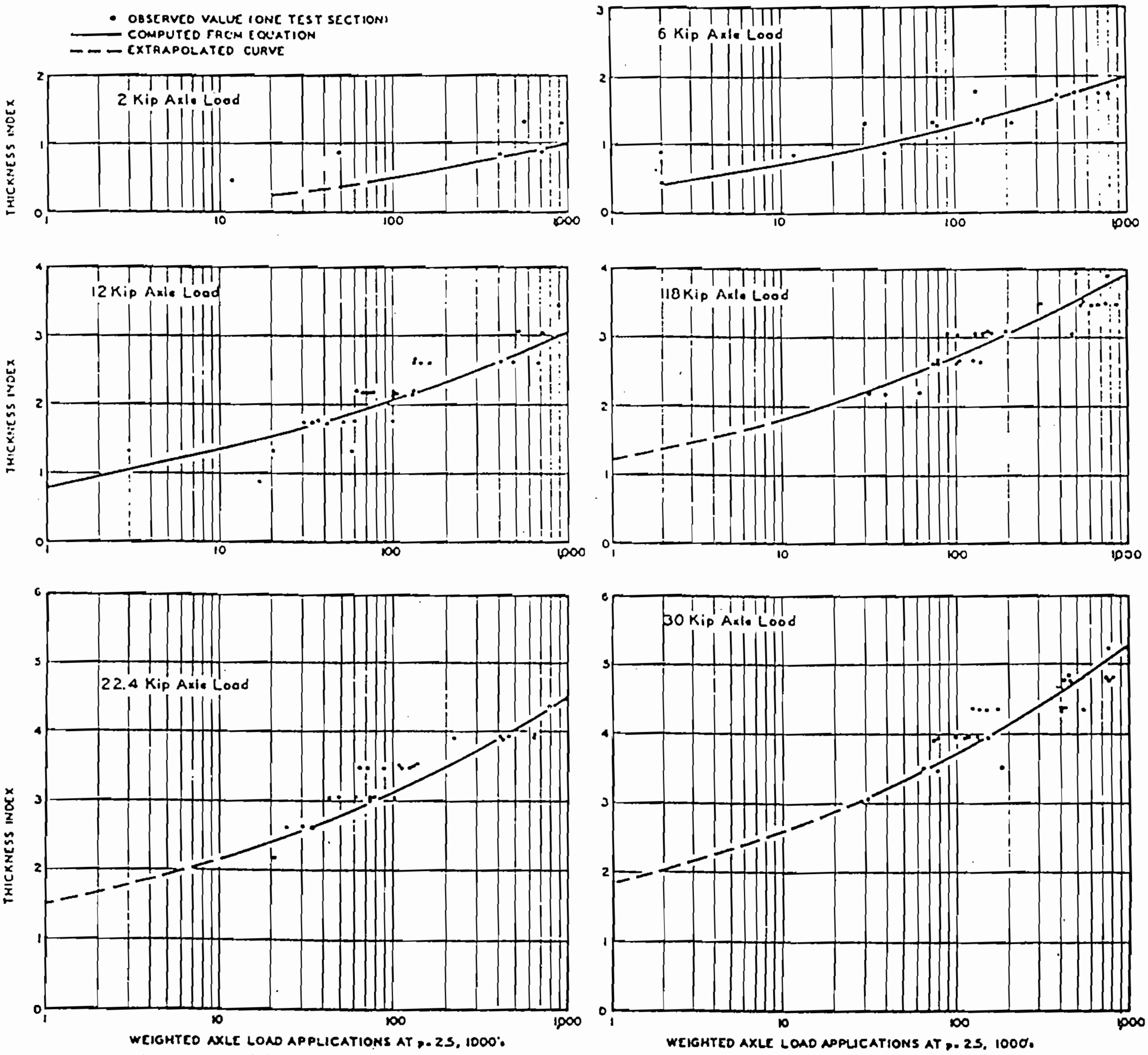

Figure 25. Main factorial experiment, relationship between design and single axle load applications at $p = 2.5$.

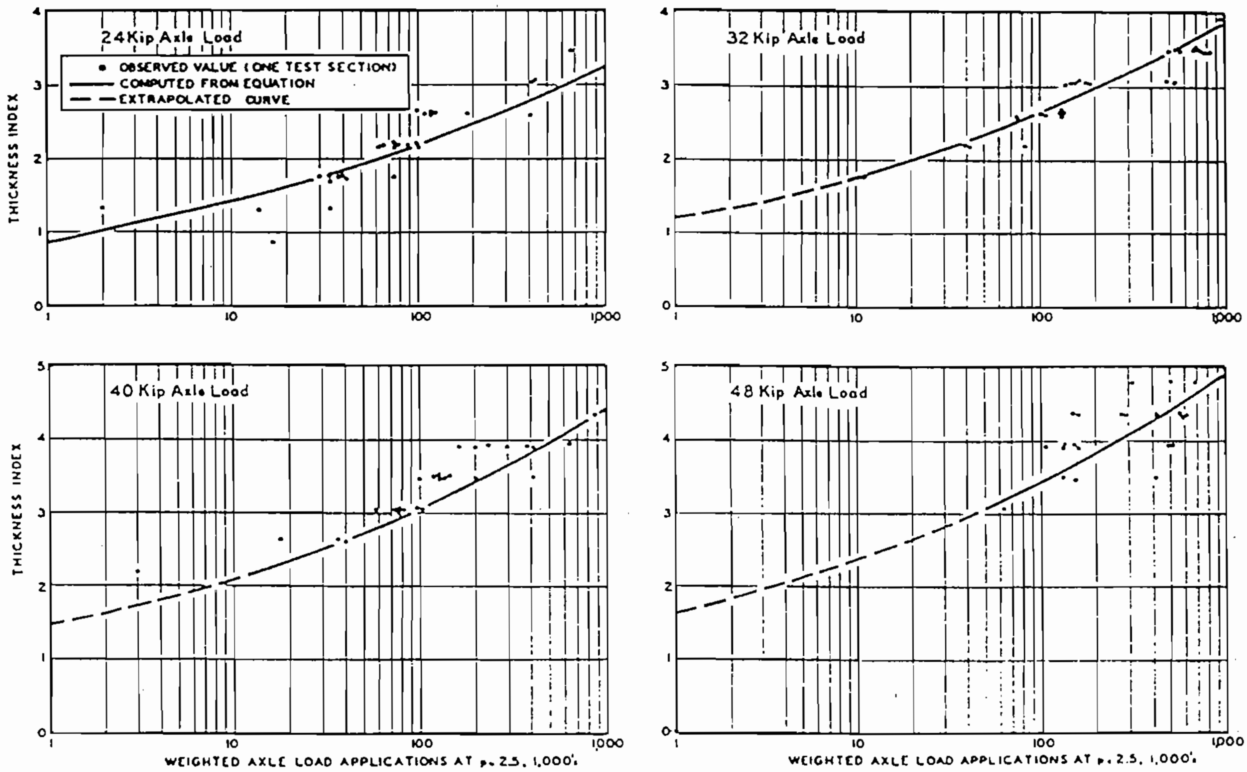

Figure 26. Main factorial experiment, relationship between design and tandem axle load applications at $p = 2.5$.

Figure 24 shows design requirements when the final serviceability value is $p = 2.5$ for a range of single and tandem axle loads at three levels of load applications. In this and the remaining graphs for flexible pavement performance (Figs. 24, 25 and 26), the final serviceability level is $p = 2.5$. The choice of 2.5 for final serviceability was arbitrary. The level of serviceability at which states actually perform major maintenance will be established by a survey of pavements scheduled for overlay or reconstruction.

Figures 25 and 26 show the correspondence between the individual curves of Figure 22 and performance data from Appendix A for each of the ten traffic lanes. Each point represents the observed number of weighted applications at which the serviceability of a test section was 2.5. Horizontal deviations of the points from the curves represent prediction errors or residuals when Eqs. 13, 14, 15, and 16 are used to predict the life of a section (to $p = 2.5$) whose design and load values are specified.

Points shown (Figs. 25 and 26) represent only those sections whose serviceability fell to 2.5 by the end of the test. All remaining sections would be represented by points whose abscissas are to the right of 1,114,000 applications. The number of such sections for any lane can be found by subtracting the number of points shown from 22 in Loop 2 and from 30 in all remaining loops. Although these sections do not appear in the graphs, their performance data were used in the development of the performance equations.

The performance data in Appendix A, Design 1, give a minimum of 5 and a maximum of 10 (p, log W) pairs for each test section. When p is fixed at 3.5, 3.0, 2.5, 2.0 and 1.5 there can be as many as 5 log W observations, and when log W is fixed at $t = 11, 22, 33, 44$ and 55 index days there can be as many as 5 observed values for p. Corresponding to each observation, log W or p, is a calculated value, $\log \hat{W}$ or $\hat{p}$, obtained from the performance equations. Differences between calculated and observed values are the residuals $\Delta \log W = \log \hat{W} - \log W$ and $\Delta p = \hat{p} - p$. Absolute values of these residuals are summarized in the first part of Table 11 which shows for each lane the number of residuals of each type as well as mean absolute residuals. Mean absolute values for $\Delta \log W$ in Loop 2, lane 1 were found to be extreme relative to the other lanes and were omitted from the grand means. Table 11 thus shows that mean values for Δp and $\Delta \log W$ were 0.53 and 0.27 for unweighted applications, and 0.46 and 0.23 for weighted applications.

Log W residuals are horizontal deviations from the performance equation curves and are thus of special interest in the use of the curves. The second part of Table 11 shows further summary of log W residuals. The co

Table D.4. Axle load equivalency factors for flexible pavements, single axles and p_t 2.5. $P_t = 2.5$

單軸

Axle Load (kips)	Pavement Structural Number (SN) 1	2	3	4	5	6
2	.0004	.0004	.0003	.0002	.0002	.0002
4	.003	.004	.004	.003	.002	.002
6	.011	.017	.017	.013	.010	.009
8	.032	.047	.051	.041	.034	.031
10	.078	.102	.118	.102	.088	.080
12	.168	.198	.229	.213	.189	.176
14	.328	.358	.399	.388	.360	.342
16	.591	.613	.646	.645	.623	.606
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.57	1.49	1.47	1.51	1.55
22	2 48	2.38	2.17	2.09	2.18	2.30
24	3 69	3 49	3.09	2.89	3.03	3.27
26	5 33	4.99	4 31	3.91	4.09	4.48
28	7.49	6 98	5.90	5.21	5.39	5.98
30	10 3	9 5	7.9	6.8	7.0	7.8
32	13 9	12 8	10.5	8 8	8.9	10.0
34	18 4	16 9	13.7	11.3	11.2	12.5
36	24 0	22.0	17.7	14.4	13.9	15.5
38	30 9	28.3	22.6	18.1	17.2	19.0
40	39.3	35 9	28 5	22.5	21.1	23.0
42	49 3	45.0	35.8	27.8	25.6	27.7
44	61 3	55 9	44.0	34.0	31.0	33.1
46	75.5	68 8	54.0	41.4	37.2	39.3
48	92 2	83.9	65.7	50.1	44.5	46.5
50	112	102.	79.	60.	53.	55.

⇑
EALF
(LEF)

EALF ⇒ X 和 ESAL 間の倍數關係
將交通量換算成同一單位

Table D.5. Axle load equivalency factors for flexible pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Pavement Structural Number (SN) 1	2	3	4	5	6
2	.0001	.0001	.0001	.0000	.0000	.0000
4	.0005	.0005	.0004	.0003	.0003	.0002
6	.002	.002	.002	.001	.001	.001
8	.004	.006	.005	.004	.003	.003
10	.008	.013	.011	.009	.007	.006
12	.015	.024	.023	.018	.014	.013
14	.026	.041	.042	.033	.027	.024
16	.044	.065	.070	.057	.047	.043
18	.070	.097	.109	.092	.077	.070
20	.107	.141	.162	.141	.121	.110
22	.160	.198	.229	.207	.180	.166
24	.231	.273	.315	.292	.260	.242
26	.327	.370	.420	.401	.364	.342
28	.451	.493	.548	.534	.495	.470
30	.611	.648	.703	.695	.658	.633
32	.813	.843	.889	.887	.857	.834
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.05	$\boxed{2.03}$	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49
50	6.12	5.76	5.03	4.64	4.86	5.28
52	7.33	6.87	5.93	5.38	5.63	6.17
54	8.72	8.14	6.95	6.22	6.47	7.15
56	10.3	9.6	8.1	7.2	7.4	8.2
58	12.1	11.3	9.4	8.2	8.4	9.4
60	14.2	13.1	10.9	9.4	9.6	10.7
62	16.5	15.3	12.6	10.7	10.8	12.1
64	19.1	17.6	14.5	12.2	12.2	13.7
66	22.1	20.3	16.6	13.8	13.7	15.4
68	25.3	23.3	18.9	15.6	15.4	17.2
70	29.0	26.6	21.5	17.6	17.2	19.2
72	33.0	30.3	24.4	19.8	19.2	21.3
74	37.5	34.4	27.6	22.2	21.3	23.6
76	42.5	38.9	31.1	24.8	23.7	26.1
78	48.0	43.9	35.0	27.8	26.2	28.8
80	54.0	49.4	39.2	30.9	29.0	31.7
82	60.6	55.4	43.9	34.4	32.0	34.8
84	67.8	61.9	49.0	38.2	35.3	38.1
86	75.7	69.1	54.5	42.3	38.8	41.7
88	84.3	76.9	60.8	46.8	42.6	45.6
90	93.7	85.4	67.1	51.7	46.8	49.7

Table D.6. Axle load equivalency factors for flexible pavements, triple axles and p of 2.5.

Axle Load (kips)	Pavement Structural Number (SN) 1	2	3	4	5	6
2	.0000	.0000	.0000	.0000	.0000	.000
4	.0002	.0002	.0002	.0001	.0001	.000
6	.0006	.0007	.0005	.0004	.0003	.000
8	.001	.002	.001	.001	.001	.001
10	.003	.004	.003	.002	.002	.002
12	.005	.007	.006	.004	.003	.003
14	.008	.012	.010	.008	.006	.006
16	.012	.019	.018	.013	.011	.010
18	.018	.029	.028	.021	.017	.016
20	.027	.042	.042	.032	.027	.024
22	.038	.058	.060	.048	.040	.036
24	.053	.078	.084	.068	.057	.051
26	.072	.103	.114	.095	.080	.072
28	.098	.133	.151	.128	.109	.099
30	.129	.169	.195	.170	.145	.133
32	.169	.213	.247	.220	.191	.175
34	.219	.266	.308	.281	.246	.228
36	.279	.329	.379	.352	.313	.292
38	.352	.403	.461	.436	.393	.368
40	.439	.491	.554	.533	.487	.459
42	.543	.594	.661	.644	.597	.567
44	.666	.714	.781	.769	.723	.692
46	.811	.854	.918	.911	.868	.838
48	.979	1.015	1.072	1.069	1.033	1.005
50	1.17	1.20	1.24	1.25	1.22	1.20
52	1.40	1.41	1.44	1.44	1.43	1.41
54	1.66	1.66	1.66	1.66	1.66	1.66
56	1.95	1.93	1.90	1.90	1.91	1.93
58	2.29	2.25	2.17	2.16	2.20	2.24
60	2.67	2.60	2.48	2.44	2.51	2.58
62	3.09	3.00	2.82	2.76	2.85	2.95
64	3.57	3.44	3.19	3.10	3.22	3.36
66	4.11	3.94	3.61	3.47	3.62	3.81
68	4.71	4.49	4.06	3.88	4.05	4.30
70	5.38	5.11	4.57	4.32	4.52	4.84
72	6.12	5.79	5.13	4.80	5.03	5.41
74	6.93	6.54	5.74	5.32	5.57	6.04
76	7.84	7.37	6.41	5.88	6.15	6.71
78	8.83	8.28	7.14	6.49	6.78	7.43
80	9.92	9.28	7.95	7.15	7.45	8.21
82	11.1	10.4	8.8	7.9	8.2	9.0
84	12.4	11.6	9 8	8 5	8.9	9.9
86	13.8	12 9	10.8	9 5	9 8	10 9
88	15.4	14.3	11 9	10 4	10 6	11 ?
90	17.1	15 8	13 2	11 3	11.6	12.1

Figure 6. AASHTO Load Equivalency Factors for Flexible Pavement (2).

Huang (1968b) compared the ESWL based on equal contact radius with that based on equal contact pressure for a variety of cases. He found that unless the pavement is extremely thin and the modulus ratio close to unity, the differences between the two methods are not very significant.

Two-layer interface deflections based on equal contact pressure were also used by the Asphalt Institute to compute the ESWL for full-depth asphalt pavements. This procedure is applicable to aircraft having less than 60,000 lb (267 kN) gross weight. By the use of Figure 2.19, simplified charts were developed for determining the ESWL for dual wheels based on the CBR of the subgrade (AI, 1973).

6.3 EQUIVALENT AXLE LOAD FACTOR

An equivalent axle load factor (EALF) defines the damage per pass to a pavement by the axle in question relative to the damage per pass of a standard axle load, usually the 18-kip (80-kN) single-axle load. The design is based on the total number of passes of the standard axle load during the design period, defined as the equivalent single-axle load (ESAL) and computed by

$$\text{ESAL} = \sum_{i=1}^{m} F_i\, n_i \qquad (6.19)$$

in which m is the number of axle load groups, F_i is the EALF for the ith-axle load group, and n_i is the number of passes of the ith-axle load group during the design period.

The EALF depends on the type of pavements, thickness or structural capacity, and the terminal conditions at which the pavement is considered failed. Most of the EALFs in use today are based on experience. One of the most widely used methods is based on the empirical equations developed from the AASHO Road Test (AASHTO, 1972). The EALF can also be determined theoretically based on the critical stresses and strains in the pavement and the failure criteria. In this section, the equivalent factors for flexible and rigid pavements are discussed separately.

6.3.1 Flexible Pavements

The AASHTO equations for computing EALF are described first, followed by a discussion of equivalent factor based on the results obtained from KENLAYER.

AASHTO Equivalent Factors

The following regression equations based on the results of road tests can be used for determining EALF:

$$\log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79 \log(18 + 1) - 4.79 \log(L_x + L_2) + 4.33 \log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \qquad (6.20a)$$

$$G_t = \log\left(\frac{4.2 - p_t}{4.2 - 1.5}\right) \tag{6.20b}$$

$$\beta_x = 0.40 + \frac{0.081(L_x + L_2)^{3.23}}{(\mathrm{SN} + 1)^{5.19} L_2^{3.23}} \tag{6.20c}$$

in which W_{tx} is the number of x-axle load applications at the end of time t; W_{t18} is the number of 18-kip (80-kN) single-axle load applications to time t; L_x is the load in kip on one single axle, one set of tandem axles, or one set of tridem axles; L_2 is the axle code, 1 for single axle, 2 for tandem axles, and 3 for tridem axles; SN is the structural number, which is a function of the thickness and modulus of each layer and the drainage conditions of base and subbase; p_t is the terminal serviceability, which indicates the pavement conditions to be considered as failures; G_t is a function of P_t; and β_{18} is the value of β_x when L_x is equal to 18 and L_2 is equal to one. The method for determining SN is presented in Section 11.3.4. Note that

$$\mathrm{EALF} = \frac{W_{t18}}{W_{tx}} \tag{6.21}$$

Equation 6.20 can be used to solve EALF. The effect of p_t and SN on EALF is erratic and is not completely consistent with theory. However, under heavy axle loads with an equivalent factor much greater than unity, the EALF increases as p_t or SN decreases. This is as expected because heavy axle loads are more destructive to poor and weaker pavements than to good and stronger ones. A disadvantage of using the above equations is that the EALF varies with the structural number, which is a function of layer thicknesses. Theoretically, a method of successive approximations should be used because the EALF depends on the structural number and the structual number depends on the EALF. Practically, EALF is not very sensitive to pavement thickness and a SN of 5 may be used for most cases. Unless the design thickness is significantly different, no iterations will be needed. The AASHTO equivalent factors with $p_t = 2.5$ and SN $= 5$ are used by the Asphalt Institute, as shown in Table 6.4. The original table has single and tandem axles only but the tridem axles are added based on the AASHTO design guide (AASHTO, 1986). Tables of equivalent factors for SN values of 1, 2, 3, 4, 5, and 6 and p_t values of 2, 2.5, and 3 can be found in the AASHTO design guide.

Example 6.7:

Given $p_t = 2.5$ and SN $= 5$, determine the EALF for a 32-kip (151-kN) tandem-axle load and a 48-kip (214-kN) tridem-axle load.

Solution: For the tandem axles, $L_x = 32$ and $L_2 = 2$, from Eq. 6.20, $G_t = \log(1.7/2.7) = -0.201$, $\beta_x = 0.4 + 0.081(32 + 2)^{3.23}/[(5 + 1)^{5.19}(2)^{3.23}] = 0.470$, $\beta_{18} = 0.4 + 0.081(18 + 1)^{3.23}/(5 + 1)^{5.19} = 0.5$, and $\log(W_{tx}/W_{t18}) = 4.79 \log 19 - 4.79 \log(32 + 2) + 4.33 \log 2 - 0.201/0.47 + 0.201/0.5 = 0.067$, or $W_{tx}/W_{t18} = 1.167$. From Eq. 6.21, EALF $= 0.857$, which is exactly the same as that shown in Table 6.4.

For the tridem axles, $L_x = 48$, $L_2 = 3$, from Eq. 6.20, $\beta_x = 0.4 + 0.081(48 + 3)^{3.23}/[(5 + 1)^{5.19}(3)^{3.23}] = 0.470$, and $\log(W_{tx}/W_{t18}) = 4.79 \log 19 - 4.79 \log(48 + 3) + 4.33 \log 3 - 0.201/0.47 + 0.201/0.5 = -0.0139$, or $W_{tx}/W_{t18} = 0.968$. From Eq. 6.21, EALF $= 1.033$, as shown in Table 6.4.

☆ 表 3-1　中山高速公路收費站年交通成長分析

單位：輛

站名	年別	全年平均日流量				站名	年別	全年平均日流量			
		小型車	大貨車	客聯車	合計			小型車	大貨車	客聯車	合計
汐止	68	12802	3792	2006	18600	楊梅	68	17641	6681	3828	28150
	69	14190	3820	2224	20234		69	18545	7566	5200	31311
	70	15417	3349	2630	21396		70	20990	7742	5153	33885
	71	17082	3152	2829	23063		71	23132	7474	5251	35857
	72	20237	3439	3059	26735		72	26105	8102	5316	39523
	73	22602	3754	3636	29992		73	29678	8680	6044	44402
	74	23959	3529	4045	31533		74	31858	8728	6736	47322
	75	27722	3843	4991	36556		75	36334	9613	7629	53576
	76	34938	4410	6424	45772		76	44564	10210	8945	63719
	77	44022	4638	7184	55844		77	55221	10652	9938	75811
	78	52756	4942	7608	65306		78	65400	11234	10649	87283
	79	61716	5626	8157	75499		79	72121	11550	10218	93889
泰山	68	34827	6574	5754	47155	造橋	68	14198	6179	3168	23545
	69	37337	7386	7610	52333		69	14659	6979	4283	25921
	70	41291	7568	7471	56330		70	16394	7049	4211	27654
	71	44687	6948	7424	59059		71	18248	6993	4399	29640
	72	52374	7687	7560	67621		72	20388	7627	4644	32659
	73	60181	8192	8404	76777		73	22685	7640	5199	35524
	74	64675	8193	9224	82092		74	24047	7688	5756	37491
	75	74329	9748	10484	94561		75	27290	8300	6566	42156
	76	91324	11295	12407	115026		76	33125	9053	7567	49745
	77	109263	12915	14197	136375		77	40858	9490	8714	59062
	78	127315	14162	14231	155708		78	48499	9780	9256	67535
	79	142159	14469	14169	170797		79	54582	10122	9528	74232

<附錄 1>

續表 3－1　中山高速公路收費站年交通成長分析

單位：輛

站名	年別	全年平均日流量				站名	年別	全年平均日流量			
		小型車	大貨車	客聯車	合計			小型車	大貨車	客聯車	合計
	68	14124	6164	3140	23428		68	8788	5691	2176	16655
	69	14518	6672	4206	25396		69	8911	6447	2941	18299
后	70	16197	6395	4115	26707	斗	70	9500	6150	3097	18747
	71	18044	6335	4292	28671		71	10950	6012	3310	20272
	72	20232	6629	4528	31389		72	12604	6557	3700	22861
	73	22445	7308	5131	34884		73	14636	6920	4200	25756
	74	23733	7449	5676	36858		74	15602	6970	4643	27215
	75	26679	8121	6504	41304		75	17664	7417	5403	30484
	76	32506	8832	7421	48759		76	21650	7910	6329	35889
里	77	39976	9338	8387	57701	南	77	27272	8171	7318	42761
	78	47057	9940	8601	65598		78	31796	8658	7649	48103
	79	53866	10531	8505	72902		79	36439	8836	7574	52849
	68	10117	5982	2200	18299		68	9078	6231	2194	17503
	69	10371	6361	3070	19802		69	9211	6948	2870	19029
員	70	11114	5855	3143	20112	新	70	9741	6649	3126	19516
	71	13076	5499	3330	21905		71	10996	6428	3335	20759
	72	15211	5816	3634	24661		72	12602	6939	3720	23261
	73	17426	6504	4150	28080		73	14825	6751	4173	25749
	74	18365	6699	4621	29685		74	15649	6620	4512	26781
	75	20914	7298	5447	33659		75	17893	7054	5311	30258
	76	25805	7962	6378	40145		76	21424	7630	6153	35207
林	77	32210	8256	7108	47574	營	77	26691	7769	6857	41317
	78	38134	8823	7302	54259		78	30420	7943	5926	44289
	79	44115	9122	7415	60652		79	35658	8116	6980	50754

〔註〕：新營站於民國78年因受莎菈颱風影響沖毀路基，曾禁行大型車一段期間進行修復，以致客聯車流量呈負成長。

續表 3－1 中山高速公路收費站年交通成長分析

單位：輛

站名	年別	全年平均日流量			
		小型車	大貨車	客聯車	合計
新市	68	11051	6503	2285	19839
	69	11422	7160	2998	21580
	70	12182	6727	3240	22149
	71	14123	6491	3417	24031
	72	15903	6841	3820	26564
	73	18041	7058	4360	29459
	74	18948	7099	4750	30797
	75	21529	7657	5593	34779
	76	26649	8419	6456	41524
	77	33537	8652	7253	49442
	78	38800	8527	7468	54795
	79	45018	8900	7939	61857
岡山	68	10804	6892	2569	20265
	69	11465	6597	3121	21183
	70	12547	5216	3295	21058
	71	14282	5130	3513	22925
	72	16334	5369	3906	25609
	73	18927	5758	4475	29160
	74	19917	5885	4905	30707
	75	23310	6397	5852	35559
	76	30034	7400	7034	44468
	77	38741	7836	7990	54567
	78	46133	7870	8371	62374
	79	52523	8074	8678	69275

(四)軸重軸次調查

Axle-load/axle-number survey

每輛大貨車標準軸重當量

The Standard EAL of Trucks

調查日期 Date		汐止站 Hsichih		后里站 Houli		員林站 Yuanlin		岡山站 Kangshan		平均 Average
次別 No	(年月日) (YY/MM/DD)	北向 N-bound	南向 S-bound	北向 N-bound	南向 S-bound	北向 N-bound	南向 S-bound	北向 N-bound	南向 S-bound	
01	70.10.13	1.330	3.304	1.993	1.063	*	*	1.422	0.581	1.616
02	71.05.12	0.403	1.367	0.091	0.976	0.051	0.586	1.088	0.389	0.619
03	71.11.17	0.580	1.074	0.643	1.197	0.056	0.531	1.062	0.716	0.732
04	72.05.10	1.501	3.553	0.761	0.803	0.056	0.658	1.048	0.435	1.102
05	72.11.16	1.214	1.344	0.619	0.886	*	*	1.728	0.716	1.085
06	73.05.16	0.044	1.320	0.513	0.020	1.428	0.013	1.470	0.432	0.655
07	73.11.14	0.643	1.599	0.500	0.521	0.931	0.737	1.500	0.670	0.888
08	74.05.14	0.300	2.090	0.612	0.747	0.990	0.650	0.920	0.810	0.890
09	74.11.17	0.593	1.126	0.504	0.657	0.905	0.615	1.360	0.560	0.790
10	75.05.14	0.296	1.246	0.658	0.677	0.973	0.504	1.036	0.774	0.771
11	75.11.14	0.413	1.386	0.059	1.102	1.053	0.773	1.120	0.554	0.808
12	76.05.13	0.529	1.700	0.994	0.007	1.125	0.915	1.301	0.907	0.935
13	76.11.18	0.897	1.025	0.759	0.770	0.980	0.840	1.398	0.777	0.931
14	77.05.10	0.570	1.037	1.007	0.672	0.799	0.698	1.091	1.376	0.906
15	77.11.16	0.190	1.080	0.790	1.140	1.050	0.967	0.801	0.714	0.842
16	78.05.17	0.449	1.606	0.694	0.972	1.167	1.031	1.452	1.250	1.055
17	78.11.15	0.448	1.253	1.020	0.860	1.037	0.905	2.697	1.151	1.171
18	79.05.16	0.322	0.860	0.585	0.343	*	1.181	0.990	0.861	0.735
19	79.11.13	0.830	0.778	0.763	0.878	0.850	0.847	1.403	1.110	0.932
20	80.05.15	0.764	1.020	0.358	1.017	0.972	0.750	1.285	0.717	0.860
21	80.11.25	0.370	1.136	*	*	*	*	1.081	1.007	0.899
22	81.05.13	0.290	0.970	1.230	1.320	1.318	1.328	1.581	1.088	1.014
23	81.11.10	0.460	0.410	1.110	1.480	1.128	0.878	1.642	0.945	1.007
平均 Average		0.584	1.404	0.737	0.813	0.881	0.765	1.325	0.806	0.914

*表示該站因地磅故或維修中，未能配合同時辦理調查

*No data

<附錄 1>

每輛聯結車標準軸重當量
The Standard EAL of Trailers

調查日期 Date 次別 No	調查日期 Date (年月日) (YY/MM/DD)	汐止站 Hsichih 北向 N-bound	汐止站 Hsichih 南向 S-bound	后里站 Houli 北向 N-bound	后里站 Houli 南向 S-bound	員林站 Yuanlin 北向 N-bound	員林站 Yuanlin 南向 S-bound	岡山站 Kangshan 北向 N-bound	岡山站 Kangshan 南向 S-bound	平均 Average
01	70.10.13	1.806	4.660	3.450	3.450	*	*	4.331	1.768	3.244
02	71.05.12	3.066	4.016	4.907	2.672	5.348	2.940	5.581	2.940	3.934
03	71.11.17	2.106	7.170	4.568	3.951	5.529	1.566	3.691	1.274	3.732
04	72.05.10	7.322	8.757	3.881	1.757	5.033	2.427	5.165	1.525	4.483
05	72.11.16	2.716	4.782	3.337	3.402	*	*	6.419	1.277	3.656
06	73.05.16	1.796	5.801	3.435	2.830	6.389	2.126	6.470	1.951	3.850
07	73.11.14	8.467	6.462	5.877	9.832	7.889	4.135	7.230	2.848	6.593
08	74.05.14	2.350	4.440	5.307	3.375	8.472	3.514	7.150	3.820	4.804
09	74.11.17	4.038	5.530	6.319	3.557	7.232	2.729	8.220	2.170	4.974
10	75.05.14	4.350	5.386	7.104	4.720	6.772	3.756	7.262	3.694	5.381
11	75.11.14	4.190	4.222	9.676	7.645	7.313	3.505	7.278	3.647	5.935
12	76.05.13	3.761	8.527	6.573	6.827	6.962	4.182	9.079	1.890	5.975
13	76.11.18	4.503	8.941	6.958	4.993	7.798	6.093	8.587	4.414	6.536
14	77.05.10	3.358	9.067	8.812	3.021	7.796	4.826	9.060	7.346	6.661
15	77.11.16	4.240	9.230	7.010	8.230	7.970	5.460	9.680	2.920	6.843
16	78.05.17	7.229	8.918	6.069	5.598	8.922	4.371	12.952	7.590	7.706
17	78.11.15	5.529	10.084	6.210	5.620	8.525	11.752	15.549	6.308	8.697
18	79.05.16	3.370	8.540	5.180	9.400	*	6.750	11.047	6.530	7.260
19	79.11.13	5.430	4.050	8.097	7.030	7.704	4.826	9.806	6.033	6.622
20	80.05.15	6.320	5.731	3.731	5.014	7.740	5.188	10.107	13.280	7.139
21	80.11.25	5.670	7.930	*	*	*	*	10.630	7.698	7.982
22	81.05.13	3.680	7.770	9.680	15.270	9.903	6.992	11.276	11.657	9.529
23	81.11.10	7.710	10.280	10.000	9.090	4.785	6.878	12.299	7.922	8.621
平均 Average		4.479	6.970	6.174	5.758	7.265	4.668	8.647	4.805	6.096

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表 3－3　中山高速公路各收費站各車種方向分佈因素

單位:%

日型	平日型			週末型			假日型		
站別 車種	小型車	大貨車	客聯車	小型車	大貨車	客聯車	小型車	大貨車	客聯車
汐止站 (方向)	50.68 (北上)	50.66 (北上)	51.83 (北上)	52.38 (北上)	52.02 (北上)	53.83 (北上)	51.95 (南下)	51.50 (南下)	52.73 (北上)
泰山站 (方向)	51.06 (南下)	54.22 (北上)	51.21 (北上)	53.22 (南下)	56.78 (北上)	53.00 (北上)	51.08 (南下)	58.77 (北上)	56.31 (北上)
楊梅站 (方向)	50.23 (南下)	51.36 (南下)	51.04 (南下)	56.10 (南下)	53.16 (北上)	53.93 (北上)	54.57 (北上)	52.54 (南下)	50.75 (南下)
造橋站 (方向)	50.29 (南下)	50.59 (南下)	52.16 (南下)	52.80 (南下)	51.02 (南下)	51.07 (北上)	56.97 (北上)	52.41 (南下)	50.22 (北上)
后里站 (方向)	50.60 (北上)	50.75 (北上)	51.94 (南下)	51.73 (北上)	56.03 (北上)	53.44 (北上)	51.89 (南下)	54.43 (北上)	50.96 (北上)
員林站 (方向)	50.20 (北上)	50.50 (南下)	51.00 (南下)	55.43 (南下)	51.68 (南下)	52.75 (南下)	54.74 (北上)	51.59 (北上)	52.54 (北上)
斗南站 (方向)	50.24 (南下)	50.09 (北上)	50.92 (北上)	52.66 (南下)	50.20 (北上)	50.17 (北上)	52.05 (北上)	52.92 (北上)	53.81 (北上)
新營站 (方向)	50.63 (南下)	51.47 (南下)	52.13 (南下)	50.10 (北上)	52.33 (南下)	51.83 (南下)	50.50 (北上)	55.75 (南下)	52.98 (南下)
新市站 (方向)	50.75 (南下)	50.84 (南下)	50.66 (南下)	51.80 (北上)	52.22 (南下)	50.65 (南下)	51.65 (南下)	52.25 (南下)	55.28 (南下)
岡山站 (方向)	50.93 (北上)	52.78 (南下)	55.39 (南下)	51.40 (北上)	50.94 (南下)	53.10 (南下)	50.80 (南下)	60.57 (南下)	50.95 (南下)

〔註〕：各日型抽樣時間分別如下：

(1) 平常日：79.11.27，79.12.17，79.12.27。

(2) 週　末：79. 4.28，79.11.17。

(3) 假　日：79.11. 4，79.12. 2。

<附錄 1>

表 3－4 中山高速公路收費站日交通組成

日型	一般日						週末						假日					
站名	小型車		大貨車		客聯車		小型車		大貨車		客聯車		小型車		大貨車		客聯車	
	輛	百分比	輛	百分比	輛	百分比	輛	百分比	輛	百分比	輛	百分比	輛	百分比	輛	百分比	輛	百分比
汐止	56456	77.80%	7730	10.65%	8379	11.55%	62788	76.87%	8230	10.08%	10658	13.05%	73063	89.13%	3674	4.48%	5239	6.39%
泰山	139447	82.10%	16077	9.47%	14322	8.43%	146571	81.83%	17235	9.62%	15309	8.55%	144203	86.35%	10026	6.00%	12775	7.65%
楊梅	66133	74.12%	12884	14.43%	10221	11.45%	75921	76.12%	13160	13.20%	10653	10.68%	92914	85.82%	6390	5.90%	8958	8.27%
造橋	45707	68.31%	11984	17.91%	9216	13.77%	56849	74.63%	10251	13.46%	9075	11.91%	67561	77.78%	9656	11.12%	9642	11.10%
后里	43758	67.99%	11838	18.39%	8764	13.62%	64268	75.78%	9696	11.43%	10850	12.79%	58904	75.65%	9590	12.32%	9371	12.03%
員林	36156	66.79%	10445	19.29%	7534	13.92%	43775	69.82%	10629	16.95%	8295	13.23%	57031	76.98%	7630	10.30%	9427	12.72%
斗南	28465	61.55%	9987	21.59%	7796	16.86%	35349	66.08%	9828	18.37%	8321	15.55%	49700	79.52%	5503	8.81%	7295	11.67%
新營	28553	62.94%	9452	20.84%	7360	16.22%	35682	67.56%	9194	17.41%	7939	15.03%	43834	81.18%	4989	8.29%	6333	10.53%
新市	37494	66.79%	10456	18.63%	8186	14.58%	44613	70.33%	10311	16.25%	8509	13.41%	56874	77.77%	8999	12.31%	7257	9.92%
岡山	44010	70.14%	9855	15.71%	8378	14.15%	53517	76.35%	8838	12.61%	7738	11.04%	71407	87.08%	4422	5.39%	6170	7.52%

表3-5 中山高速公路收費站日平均交通量

站名	小型車		大貨車		客聯車	
	輛	百分比	輛	百分比	輛	百分比
汐止	60497	79.98%	7034	9.30%	8110	10.72%
泰山	141361	82.85%	15098	8.85%	14170	8.30%
楊梅	72624	76.96%	11696	12.39%	10044	10.64%
造橋	51426	71.43%	11297	15.69%	9276	12.88%
后里	49543	70.95%	11108	15.91%	9176	13.14%
員林	41188	69.66%	9939	16.81%	8000	13.53%
斗南	33460	66.45%	9117	18.11%	7776	15.44%
新營	33403	67.86%	8572	17.41%	7248	14.72%
新市	42172	69.84%	10160	16.82%	8056	13.34%
岡山	50544	74.96%	8683	12.88%	8204	12.17%

表3-6 中山高速公路收費站日交通量抽樣誤差

站名	抽樣值	實際值	抽樣誤差
汐止	75642	75500	0.19%
泰山	170629	170797	0.10%
楊梅	94363	93889	0.50%
造橋	71999	74233	3.01%
后里	69827	72902	4.22%
員林	59127	60652	2.51%
斗南	50353	52849	4.72%
新營	49222	50754	3.02%
新市	60388	61857	2.37%
岡山	67430	69275	2.66%
平均誤差值：2.33%			