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Problems Encountered in Using Vehicle Ride as a Criterion of Pavement Roughness

B.E. QUINN

One criterion of pavement roughness has been the ride experienced in a passenger vehicle. This ride depends on the properties of the vehicle as well as those of the pavement. Recent changes in vehicle design (less weight and front-wheel drive) may affect this pavement criterion. To show the relationship between vehicle properties and ride, a simple mathematical model was selected for the vehicle. Vertical vehicle acceleration was used as a measure of the ride. For a pavement of known properties, the ride was determined for different speeds and different vehicle suspension characteristics. Significantly different values for the pavement criterion were obtained for the same section of pavement. It is believed that ride can still be used as a criterion of pavement roughness but that operating speeds and vehicle characteristics must also be considered in establishing this criterion.

For many years the ride experienced by the occupant of a vehicle has been used as a criterion of pavement roughness. A person traveling over a section of pavement in a passenger vehicle at a selected speed subjectively evaluates the experience. This evaluation is accepted as a criterion of the roughness of the pavement section in question.

In spite of the subjective nature of this procedure it has worked well for several years largely because passenger vehicles have been similar in design over this period of time. Moreover, cars used to be heavier and the weight distribution resulted in almost the same wheel loads on the front and rear wheels.

Within the past 5 years significant changes have been made in the design of passenger cars. The weight has been reduced in some models so that the weight of the passenger is now a higher proportion of the total weight. Of greater importance is the fact that with front-wheel drive the front wheels carry more of the total weight when only the front seat is occupied. In addition, the overall length of these vehicles has been reduced. As a result of these and other changes, there is a difference in the riding properties of the newer cars. Thus, it is possible to have different pavement roughness criteria for the same pavement if different vehicles are used to evaluate the ride.

In addition, the response of the vehicle to the pavement varies with vehicle speed. Depending on the properties of the pavement, it is possible to improve the ride by selecting the proper vehicle velocity. In general, the effects of short-wavelength disturbances can be minimized by traveling at higher velocities whereas long-wavelength disturbances become more objectionable at higher speeds.

The Kentucky Department of Highways has evaluated the riding quality of highways by measuring the acceleration experienced by a person riding in a vehicle (1, p. 14). A difference of 8 percent was observed between tests conducted with a full tank of gasoline and one conducted with a tank that was nearly empty. It is thus possible for vehicle properties to change measurably during the operation of the vehicle.

The purpose of this paper is to outline a method by which vehicle properties can be described in a meaningful way but the basic concept of evaluating a pavement based on ride can still be used.

DESCRIBING RIDE

The Kentucky Department of Highways used triaxial passenger acceleration as a criterion of ride. In this paper only vertical acceleration is considered. A more accurate description of ride requires a consideration of the allowable levels of acceleration at various frequencies.

DESCRIBING THE VEHICLE

A simple model of the vehicle is used (see Figure 1). The single wheel of the model is assumed to have no weight and to experience a vertical displacement y. This displacement is produced by the highway profile. To the wheel is attached the lower end of a linear spring of stiffness k that represents all the stiffness in the suspension system of the vehicle. In a similar way, all of the damping is represented by a linear shock absorber that has a damping constant c and is also attached to the wheel. To the upper ends of the spring and the shock absorber is attached a mass m that represents all of the sprung mass of the vehicle. The vertical acceleration of this mass is considered as the acceleration experienced by the passenger and, hence, the ride criterion.

The vehicle characteristic used in this analysis is the ratio of the passenger acceleration divided by the vertical displacement of the wheel plotted as a function of frequency (see Figure 2). This characteristic represents the application to a highway problem of a frequency response technique used in automatic control problems to describe a system of interest (2, p. 72). This technique requires that the input and the output of a given system be defined. A sinusoidal input is then applied at a selected frequency to produce a sinusoidal output. The ratio of the amplitude of the output to that of the input is then determined at the selected frequency to give one point on what is known as a frequency response curve.

This process is repeated until the curve is defined over the range of frequencies of interest. This curve describes the system under consideration

Figure 1. Simple model of a passenger vehicle.

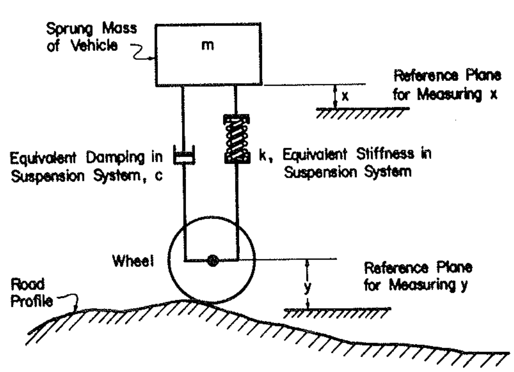


Figure 2. Vehicle characteristics used to predict ride.

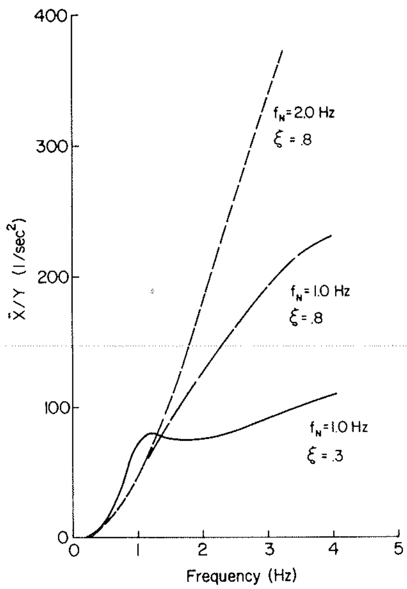
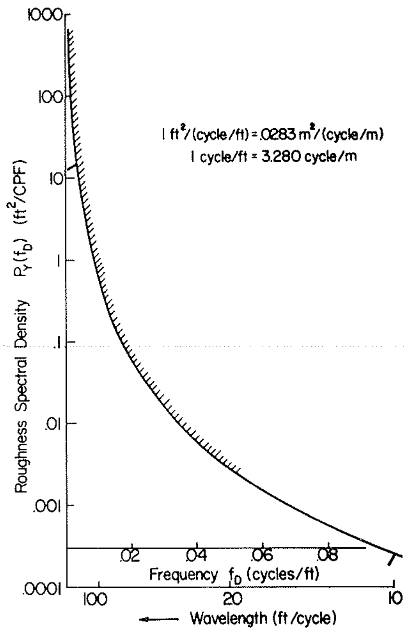


Figure 3. Pavement roughness spectrum.



and has many useful applications. By using the appropriate input and output quantities, curves of this type have been used to predict dynamic tire forces (3) and to determine pavement roughness spectra from vehicle motion (4). A response characteristic can be determined either mathematically or experimentally.

In this case the system under consideration is the vehicle. The input is defined as the displacement y at the wheel resulting from travel over the pavement surface. The output is the ride $\ddot{X}$, which has already been discussed. For this situation the frequency response characteristic has been computed mathematically, but it can also be determined experimentally, as has been done for actual passenger cars (5). The amplitude of the sinusoidal input displacement is represented by Y, and the amplitude of the sinusoidal output acceleration is represented by $\ddot{X}$.

The ratio of interest is $\ddot{X}/Y$, which is shown in Figure 2 for three different vehicle suspension conditions. The mathematical development of $\ddot{X}/Y$ is omitted here to conserve space and also because it is believed that a serious study of this problem should involve an experimental determination of actual vehicle properties.

The curves in Figure 2 show the effect on the $\ddot{X}/Y$ characteristic of the natural frequency of the vehicle (f_N) and also of the damping in the suspension system as indicated by the damping ratio (ξ). In general, larger values of ξ represent more highly damped suspension systems. The units of $\ddot{X}/Y$ are those of acceleration divided by displacement, which reduce to 1/sec².

DESCRIBING THE PAVEMENT

A pavement roughness spectrum is used to describe the pavement (6). This curve, shown in Figure 3, indicates the extent to which various wavelengths are present in the pavement profile. The ordinates of this curve represent the roughness spectral density in square feet per cycle per foot, and the abscissae represent the reciprocals of the wavelengths in units of 1/ft. The area under this curve represents the mean square value of the roughness in square feet. The reciprocal of the wavelength is referred to as a distance-based frequency (f_D) in contrast to the time-based frequency (f) used in describing the vehicle. [Information for Figure 3 was obtained by using instrumentation (7) calibrated in the units shown.] This description of the pavement depends only on the profile and is thus geometric in nature.

The question might be raised as to why the area under the pavement roughness spectrum could not be used as a criterion of pavement roughness. The reason is that not all wavelengths affect the riding qualities of a pavement. This is particularly true of long wavelengths, which usually make large contributions to the mean square value of the roughness. Going up over a hill and down into a valley introduces an enormous wavelength into the pavement profile, but if the pavement is free of very short wavelength distortions the ride will not be adversely affected. In this way ride serves to identify pavement distortions that are important to the user.

A pavement roughness spectrum can be computed from elevation measurements (6) or determined from the data obtained from any device that measures the pavement profile. It can also be measured directly by using special equipment (7). The advantage of using this description of the pavement is that it can be combined with the vehicle frequency response previously mentioned to predict the ride, which can then be used as a criterion of pavement roughness.

PROCEDURE FOR PREDICTING RIDE

Because the pavement is described in terms of cycles per foot, it is necessary to convert these units to cycles per second (hertz) in order to be able to combine the pavement roughness spectrum with the vehicle frequency response characteristic. To do this it is necessary to know the velocity (V) at which the vehicle is moving over the pavement. If V is known, the ordinates and the abscissae of the roughness spectrum can be transformed to produce a new roughness spectrum curve expressed in terms of a time-based frequency. This transformation can be achieved as follows:

$$f = V \times f_D \qquad (1)$$

$$P_Y(f) = P_Y(f_D)/V \qquad (2)$$

where

f = time-based frequency (cycles/sec or Hz),
f_D = distance-based frequency (cycles/ft),
$P_Y(f_D)$ = pavement roughness spectral density [ft²/(cycle/ft)], and
$P_Y(f)$ = transformed pavement roughness spectral density [ft²/(cycle/sec)].

The results of this transformation are shown in Figure 4, where two different velocities have been used as indicated and two separate and distinct curves have been obtained. This indicates that different operating speeds will result in different inputs to the vehicle and, hence, different vehicle outputs can be expected. The effects of the two velocities can also be seen in Figure 3, where that portion of the curve designated by the closely spaced tic marks above the curve is associated with the higher velocity and that portion lying within the two longer tic marks below the curve is associated with the lower velocity. Figure 3 shows that long wavelengths become more significant in the input to the vehicle at high velocities whereas shorter wavelengths become less significant. In terms of the input, the vehicle is exposed to different highways at different speeds even though the same section of pavement is involved, which introduces problems when ride is used as a criterion of pavement roughness.

It is possible to determine the mean square value of the passenger acceleration by using the appropriate information in Figures 2 and 3. This requires the use of the following relationship (8, p. 197):

$$P_{\ddot{X}}(f) = P_Y(f) \times (\ddot{X}/Y)^2 \qquad (3)$$

where $P_{\ddot{X}}(f)$ is the acceleration spectral density [(ft²/sec/sec)²/Hz].

Acceleration spectral density curves are shown in Figure 5 for the vehicle characteristic shown by the solid line in Figure 2 for velocities of 27.3 mph (12.2 m/sec) and 54.6 mph (24.4 m/sec). The area under each curve represents the mean square value of acceleration experienced by the passenger at the velocity indicated. This is the criterion of pavement roughness mentioned previously.

Because the mean square value of acceleration is the variance of acceleration, the standard deviation can be obtained by taking the square root of the variance. Tests (9, p. 129) have indicated that in an acceleration time record the amplitudes can be approximated by a normal distribution that has a zero mean. Once the standard deviation is available, it is possible to estimate the probability of encountering various magnitudes of acceleration and thus evaluate the ride in a more meaningful way.

Figure 4. Transformed pavement roughness spectra.

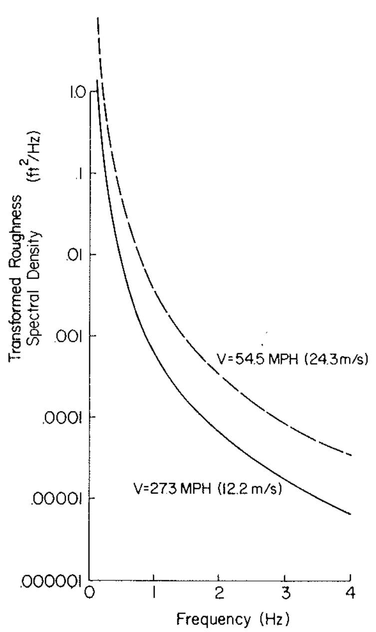
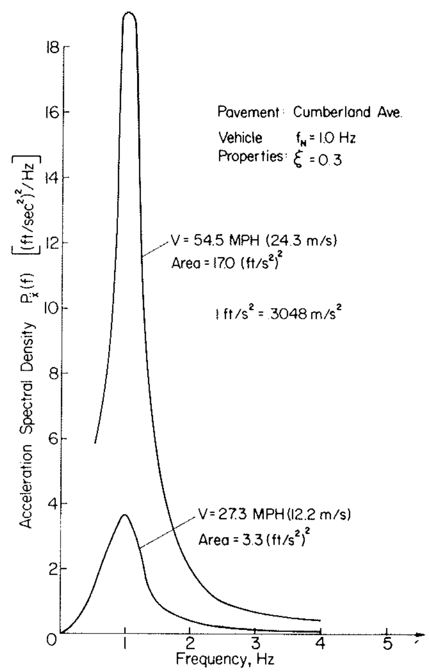


Figure 5. Acceleration spectral densities for two speeds.



EVALUATING A LOCAL PAVEMENT

To illustrate the problems encountered, the roughness spectrum of a convenient pavement section was measured by using a modified BPR roughometer (7). This pavement was used daily but was generally considered to be rougher than average. The roughness spectrum, shown in Figure 3, was transformed to a time-based frequency for two different velocities, as shown in Figure 4.

Vehicles with various suspension properties were then investigated for natural frequencies ranging from 1 to 2 Hz. The lower frequency represents a vehicle with a soft suspension and, hence, a better ride. This is typical of heavier, higher-priced vehicles, in which ride is an important characteristic. The 2-Hz frequency is more characteristic of stiffly sprung vehicles such as those in which handling is most important. Damping ratios from 0.8 to 0.3 were selected to represent shock absorbers in good to poor condition.

The standard deviations of the accelerations were computed for different speeds, natural frequencies, and damping ratios (see Figure 6). The interior of the parallelogram in Figure 6 approximates the region of possible values for the standard deviations of passenger acceleration. Values ranging from 1.8 to 6.9 ft/sec² (0.55 to 2.1 m/sec²) are indicated.

By selecting a vehicle with a natural frequency of 2 Hz and a damping ratio of 0.3 (the upper boundary of the region shown in Figure 6), standard deviations ranging from 3.1 to 6.9 ft/sec² (0.96 to 2.1 m/sec²) can be obtained on the same pavement by varying the speed from 27.3 to 54.5 mph (12.2 to 24.4 m/sec). The statistical significance of this is shown in Figure 7, where (assuming a normal distribution) the probability density curves are plotted versus acceleration for the two velocities under consideration. Broken vertical lines are shown at 0.1 _g_ (3.21 ft/sec/sec) and at -0.1 _g_. The probability of experiencing accelerations outside these limits is represented by the area bounded by the curve of interest that lies outside these vertical lines. At 27.3 mph this probability is 0.317; at 54.5 mph it is 0.617.

Figure 6. Standard deviation of acceleration as affected by vehicle properties and vehicle speed.

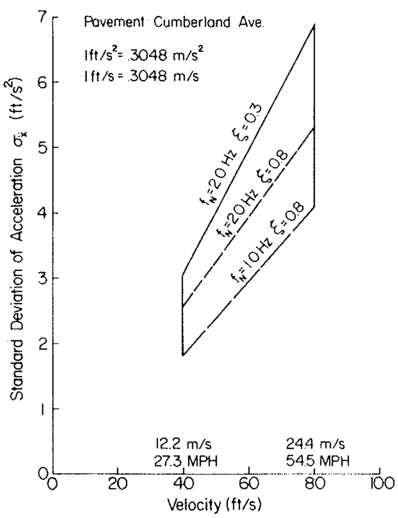
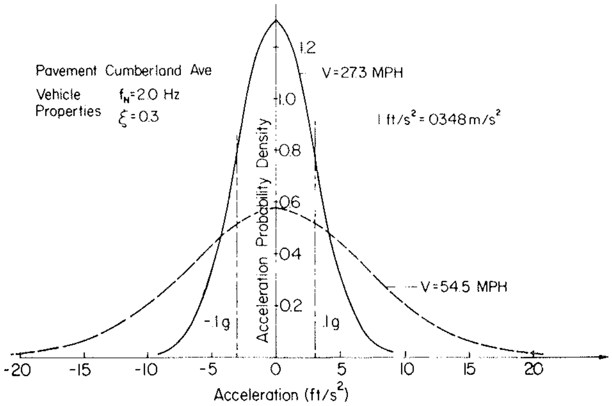


Figure 7. Probability density functions for accelerations experienced by a vehicle at two different speeds on the same pavement.



CONCLUSIONS

For the simple vehicle model used in this paper, a wide range of passenger accelerations was obtained even though the identical section of pavement was being evaluated in each case.

In the past, vehicle ride has been a satisfactory criterion for evaluating pavement roughness because there was relatively little change in the design of the most commonly used passenger vehicles. In recent years, however, many passenger vehicles have undergone extensive design changes. It is believed that the ride in these newer vehicles can still be used to evaluate pavements but that before this is done a careful study of the effect of the properties of the newer vehicles on ride should be undertaken. In addition, it is also desirable that certain standard conditions be established to obtain a measurement of pavement roughness that will reflect only the pavement properties.

REFERENCES

1. R.L. Rizenbergs. Accelerometer Method of Riding-Quality Testing. Kentucky Department of Highways, Frankfort, Rept. KYHPR-64-25, Part 2, 1965.
2. K. Ogata. Modern Control Engineering. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1970.
3. B.E. Quinn and D.R. Thompson. Effect of Pavement Condition on Dynamic Vehicle Reactions. HRB, Bull. 328, 1962.
4. T.D. Gillespie, M.W. Sayers, and L. Segel. Calibration of Response-Type Road Roughness Measuring Systems. NCHRP, Rept. 228, 1980.
5. B.E. Quinn and D.E. Gevers. Laboratory Measurement of Vehicle Characteristics for Predicting Highway Behavior. FHWA, Rept. FHWA-RD-75-2, 1975.
6. B.G. Hutchinson. Analysis of Road Roughness Records by Power Spectral Density Techniques. Ontario Department of Highways, Ontario, Canada, Rept. 101, 1965.
7. B.E. Quinn and L.M. Smeyak. Measuring Pavement Roughness Spectra Using the Modified BPR Roughometer with Additional Refinements. FHWA, Rept. FHWA-RD-72-27, 1972.
8. J.H. Lanning and R.H. Battin. Random Processes in Automatic Control. McGraw-Hill, New York, 1956.
9. B.E. Quinn and E.W. Jones. Relating Pavement Roughness to Vehicle Behavior. FHWA, Rept. FHWA-RD-75-1, 1975.

Evaluation of Panel Rating Methods for Assessing Pavement Ride Quality

J.B. NICK AND M.S. JANOFF

The results of a pilot study that attempted to determine the preferred psychophysical scaling method for obtaining panel ratings of pavement ride quality are reported. Five principal tasks were undertaken: site selection, scale selection, panel selection, design and conduct of the experiment, and analysis and interpretation of the results. Thirty-three flexible road sections covering a wide range of roughness (uniform within each site) were selected, and a 92-mile route that could be traversed, at normal operating speeds, in 3.5 hr was established. Three candidate scales were selected for evaluation: the original AASHO rating scale, a scale developed by Holbrook that uses precisely defined and positioned word cues, and a nonsegmented scale with word cues only at the end points. Fifty-four average drivers were divided into 3 panels and rated the 33 sites in groups of 3. Each panel member used only one of the 3 scales and rated each section only once. A Mays ride meter was used to obtain roughness measurements for each section. The analysis was designed to (a) determine which scaling method resulted in the greatest agreement between raters and (b) determine which of the three scaling methods resulted in the best correlation between subjective and objective measures of road roughness. The results indicated that either subjective scale, if carefully used (i.e., with exact instructions and precise control of conditions), can provide high agreement between raters and exceptionally high R^2 values.

In this paper, the results of a pilot study conducted to evaluate three psychophysical scaling methods for obtaining panel ratings of pavement ride quality are summarized. The pilot study was part of a larger project conducted by Ketron, Inc., for the Pennsylvania Department of Transportation (PennDOT) for the purpose of investigating the predictive relationship between subjective ratings of pavement ride quality and Mays ride meter (MRM) measures of pavement roughness. The overall goal of the entire project was to develop regression equations between the subjective and objective measures--one for each of the three types of surfaces (flexible, rigid, and flexible over rigid)--so that MRM measurements could be used as surrogates of subjective responses to road roughness.

As a first step in meeting the objectives of the project, a pilot study was designed to evaluate several candidate rating scales for assessing pavement ride quality. The pilot study consisted of five interrelated tasks:

1. Selection of candidate scaling methods,
2. Selection of sites to be rated,
3. Selection of the rating panels,
4. Design and implementation of the experiment, and
5. Analyses and interpretation of the results.

The results of these tasks are summarized in this paper.

SELECTION OF RATING METHODS

Direct Versus Indirect Scaling Methods

Because the overall goal of this study was to establish the quantitative relationship between physical measurements of pavement roughness and subjective perceptions of ride quality, it was necessary to measure the psychological experience on at least an interval scale. That is, to establish a functional relationship between two measures, certain mathematical operations would be required that can only be meaningfully conducted when both quantities are measured on at least an interval (or ratio) scale. A problem arises, however, in that externally observable judgments by human subjects must be relied on to obtain the ratings of ride quality. Thus, certain assumptions must be made about the ability of the subjects to judge pavement ridability at the desired level of measurement.

There are two basic methods for obtaining ratings on an interval scale: direct and indirect. The fundamental difference between these methods is the assumption about the ability of the subject to describe preferences or sensations at the intended level of measurement. Direct scaling refers to methods of obtaining judgments of psychological quantities directly on an interval (or ratio) scale. In these methods, the desired quantitative level of judgment can be obtained either by carefully designing the scaling instrument or by the instructions the experimenter gives the individual raters at the beginning of the experiment. If it is assumed that the subjects have been able to carry out the task as intended, then the scale values of the stimuli (roads in this case) on an interval scale are given directly by their ratings and the scaling problem becomes one of merely averaging the results.

For indirect scaling methods, a subject's response, whether it be a statement of preference or an intensity of sensation, is only considered to be an indirect index of a mechanism mediating between the hypothetical psychological responses and the magnitudes of the external stimuli. Thus, in these methods an investigator assumes that, if a number of subjects were asked to judge each of several stimuli as belonging in one of a limited number of categories, only a rank ordering (i.e., an ordinal level of measurement) of preferences of road sections would be obtained, and additional statistical methods must be used to obtain the final ratings on an interval level of measurement.

Three scaling methods were selected for evaluation in the pilot study. One method was intended to yield indirect scale values and the other two were intended to yield scale values directly (although each was eventually analyzed as if it yielded both direct and indirect values).

All of these methods involve the use of a graphic rating-scale technique to record subjects' responses. This technique is a special type of category scaling that, in one form or another, has been used in almost all serviceability studies. Depending on the underlying assumptions, graphic rating techniques such as the one used here allow one to treat the responses as either lying in one of a limited number of categories (indirect methods) or lying along a linear continuum of unlimited categories (direct methods).

An entirely different approach, and one recommended by Holbrook (1), would be to ask the subjects to match a number directly to the perceived magnitude of ride quality and eliminate all intervening categories, orderings, cues, comparisons, and theories. However, magnitude estimation, as this method is called, was eliminated from consideration because there was some question as to how well the subjects would be able to make the successive ratio judgments required by this method over the 100 roads in two days that the main study would involve.

WEAVER-AASHO SCALE

The first scale considered for evaluation was what is referred to here as the Weaver-AASHO scale. This scale, shown in Figure 1a, was originally used for the AASHO Road Test by Carey and Irick (2). Because this scale and similar versions of it have been used in a number of studies, it was selected for evaluation primarily as a baseline instrument with which the other scales could be compared.

This scale was also intended to be the one used to obtain indirect scale values by using an analysis method adopted by Weaver. After the AASHO Road Test, reviews such as that by Hutchinson (3) noted that the method used by Carey and Irick (2) may have violated several principles of psychometric methods. The most notable violations were the following:

1. The systematic errors of leniency (the tendency of a subject to rate too high or too low, for whatever reasons), the halo effect (contamination of subjective response caused by stimulus attributes other than those under consideration), and central tendency (the tendency of subjects to hesitate in giving extreme ratings) were not compensated for or removed from the raw data.
2. The raters were not provided with descriptive cues, or anchors, that corresponded closely enough to the trait being measured (i.e., ride quality).
3. It was assumed that the raters were good instruments of quantitative observation and yielded ratings directly on an interval scale.

There is considerable debate in the literature about the last issue. Many researchers believe that, in this situation, subjects are only capable of rendering judgments on an ordinal level and not directly on an interval level; that is, because it is possible that the judgments are only rank ordered, it is possible to determine whether one road was judged better or worse than another but not by what magnitude. Because of this mistrust of the ability of the subjects to make their judgments at the desired level of measurement, Weaver (4) of the New York State Department of Transportation (NYSDOT) adopted an indirect scaling method called the method of successive categories to analyze the data obtained by this scale. This method is based on Thurstone's law of comparative judgment and was developed by Guilford (5).

Although the method of successive categories relies heavily on untestable, hypothetical models of human judgment and requires a complicated analysis procedure, it was selected as a candidate method for evaluation in the pilot study for two reasons:

1. No study had yet been conducted that compared the results obtained by the more sophisticated direct scaling methods with the results from an indirect method.
2. Because Weaver had been using this method in New York for several years, apparently with success, a potential comparative data base was provided.

However, even with the implementation of the method of successive categories to transform ordinal judgments into interval-level scale values, this scale still potentially violates the other two principles of psychometric methods mentioned earlier by not providing more descriptive cues and by not compensating for the presence of possible systematic errors in the data.

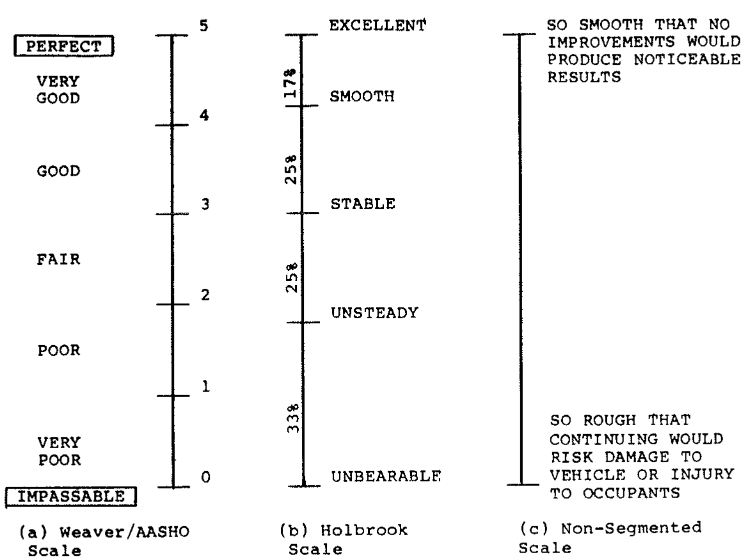
<u>Holbrook's Scale</u>

It is theoretically possible both to overcome the potential problems inherent in the Weaver-AASHO scale and to create a scale that yields interval-scaled ratings directly through more careful selection and placement of relevant cue words along a continuum. Holbrook (1) devised such a scale after considerable experimental work in selecting and placing the cue words (see Figure 1b).

The cue words were selected by having 80 subjects rate 92 words that could be used to describe varying degrees of ride quality along an 11-point roughness continuum. The 5 words shown on the scale were selected because of their narrow distribution in this preliminary scaling task. The median values for these 5 words along the 11-point roughness scale determined the location of the final scale.

This scale is presumably an improvement over the Weaver-AASHO scale because the cue words, or an-

Figure 1. Scales tested in pilot study.



chors, are not only relevant to the task of evaluating ride quality but are also placed along the scale so that the intervals between them represent similarly spaced intervals along the psychological roughness continuum. In addition, because of these characteristics, the ratings obtained with this scale should be relatively free of the systematic errors of leniency and central tendency. Both types of errors are generally counteracted by anticipating their magnitude and direction and then adjusting the position of the cue words along the scale to bias the raters so that they will not make these errors. However, because the strength and the position of the descriptive adjectives for this scale have been empirically predetermined, the compensating adjustments should have been made automatically. (The halo effect must still be dealt with by giving specific instructions to the raters.)

Nonsegmented Scale

Even though Holbrook's scale seems to represent a considerable improvement over the Weaver-AASHO scale, there could still be problems with the use of intermediate cue words. As Torgerson (6) notes, "In general, when we attempt to construct a scale for a set of stimuli using one of the subjective estimate methods, we are free to specify two and only two points on the continuum." If more points are to be defined, one must be sure that the placement of these points and their associated cues will represent comparably spaced intervals along the psychological scale for all (or almost all) of the raters. Although Holbrook was careful in selecting and placing his cues, the meaning of the intermediate cue words--"smooth," "stable," and "unsteady"--could still be vastly different for many subjects.

To avoid the connotative problems associated with the intermediate cue words, a better scale may be one in which only the ends are anchored and the subjects are allowed to place their rating marks on the scale unaided, as Torgerson has recommended. Hence, the last scale to be evaluated in the pilot study was what is referred to here as a nonsegmented scale (see Figure 1c). This scale was intended to yield direct interval values by virtue of the instructions given to the subjects. The subjects were instructed to place their marks on the scale so that the ratios of the differences in the distance between their marks reflected similar differences in the ride quality of the roads.

SITE SELECTION

The original plan for site selection required that 24 flexible-pavement sites be used in the pilot study. These 24 sites were to represent (a) four of the five maintenance functional classifications (MFCs) into which PennDOT stratifies the roads in the state (MFC-A was not represented because there are few flexible pavements in this class), (b) two different topographies (flat and rolling or hilly), and (c) three degrees of roughness spanning a broad range within each MFC.

However, in addition to the MFC, topography, and roughness requirements, there were other site characteristics that were considered essential to a successful study and eventually became important controlling factors in the evolution of the final course. In order of descending importance, these characteristics were the following:

1. The sites were to be as close as possible to each other to minimize the travel time between sites and through the entire course. Because it was anticipated that two runs a day would be made, a maximum total travel time of 3 hr was considered essential.

2. The length of the sites was to be adjusted so that travel time through each site would be equal and raters would thus have the same amount of time to evaluate the ride quality of each site. Thus, the length of each site depended on the expected speed to be used in driving through the site.

3. Each site was to have a uniformity of roughness over its entire length. In addition, approximately 500 ft of roadway before and after each site was to exhibit the same roughness characteristics. It was believed that this would help the raters avoid any uncertainty as to what the rating should be if an unrepresentative surface or bump were encountered.

4. The sites were to be selected only from rural areas in order to control the surrounding environment and minimize the possibility of being stopped within a site because of congestion or traffic lights.

5. Ideally, the sites were to be as straight as possible; if they were curved, the curves were to be gentle enough to allow speed to be maintained.

Initial field reconnaissance of potential sites on which PennDOT had previously used the Mays ride meter revealed that a significant number of the sites did not meet these criteria or their tabulated roughness had changed as a result of resurfacing. Consequently, the site selection strategy consisted of mapping a convoluted route linking the few usable sites and then selecting sites along this route that appeared, by visual inspection, to meet the requirements. In addition to the minimum of 24 sites required by the initial design, to prevent rater boredom other sites were selected along the route so that the time between sites was not excessive.

After the candidate sites and the route were approved, MRM measurements were made by PennDOT. All sites and almost all of the intersite roads were measured at a nominal speed of 40 mph. The PennDOT standard operating procedure is to measure all roads at 40 mph (or 25 mph if absolutely necessary) because the PennDOT MRMs have been calibrated at these speeds and correlation factors are available for comparison of the results obtained when either of these two speeds is used.

The resulting roughness distribution was found to be fairly rectangular and ranged from a low of 94 in./mile to a high of 752 in./mile. Furthermore, when linked, these sites formed a 92-mile route that took a manageable 2.75 hrs to drive at the posted speed limits. The median time between sites was 3.5 min.

The characteristics of the sites are given in Table 1. It should be noted that the ultimate distribution of exposure times ranged from 18 to 34 sec and the median time was 27.5 sec. Ideally, the length of a site was to be the empirically determined distance that yielded the desired exposure time, which, according to the original design, could have been anywhere between 23 and 29 sec. Frequently, however, the actual end point was picked to coincide with the most convenient vertical object nearest to the ideal end point that could be painted so as to be seen easily by the driver. Hence, the distribution was somewhat broader than desired. Nevertheless, no evidence was found to suggest that the differences in exposure time had a significant impact on the outcome of the study.

Table 1. Summary statistics for pilot study sites.

Site	MFC	Test Speed (mph)	Length: Miles	Length: Feet	Travel Time (sec)	Axle Displacement: Total (in.)	Axle Displacement: Inches per Mile
1	B	45	0.308	1,626	25	131	424
2	C	45	0.426	2,249	34	76	17
3	D	30	0.291	1,536	35	219	752
4	D	35	0.285	1,505	29	161	56
5	E	30	0.245	1,204	29	102	418
6	E	35	0.200	1,056	21	60	301
7	D	40	0.400	1,440	25	95	238
8	D	55	0.390	2,059	25	42	108
9	E	35	0.300	1,584	31	108	358
10	D	45	0.432	2,281	33	52	120
11	E	40	0.372	1,964	34	100	268
12	B	45	0.364	1,922	29	40	109
13	B	45	0.272	1,436	22	33	121
14	D	40	0.252	1,331	23	28	109
15	C	45	0.299	1,579	23	39	130
16	C	55	0.400	2,112	26	38	94
17	D	35	0.324	1,711	33	209	645
18	E	35	0.335	1,769	34	97	290
19	E	40	0.249	1,315	22	51	203
20	E	35	0.209	1,104	22	45	216
21	E	30	0.208	1,098	25	61	295
22	E	35	0.285	1,505	29	111	390
23	D	40	0.274	1,447	25	63	231
24	D	30	0.257	1,357	31	158	615
25	D	45	0.228	1,204	18	30	133
26	D	40	0.331	1,748	30	85	257
27	E	30	0.204	1,077	24	149	733
28	D	45	0.294	1,552	24	50	170
29	B	40	0.292	1,542	26	72	248
30	D	40	0.275	1,452	25	133	485
31	D	40	0.272	1,436	24	116	428
32	C	45	0.389	2,054	31	56	145
33	B	45	0.369	1,948	30	76	207

SELECTION OF PANEL MEMBERS

Panel Size

In the pilot study three panels of 18 observers were used, one panel to evaluate each of the three scaling methods. This panel size was selected as a conservative compromise between the requirement of 20, to keep the maximum error in estimating the true population mean rating for any given site at or below 0.4 scale unit on the assumption that the sampling distribution was not normal, and the requirement of 9 observers for the same error if the sampling distribution was assumed to be normal. Ordinarily, smaller panel sizes would be acceptable because it would not be unreasonable to assume that the distribution of sample means approaches normality. However, there were at least two reasons for using more conservative estimates. First, because it could not be guaranteed a priori that the obtained ratings would be on an interval level of measurement, the estimated standard deviation of 0.6 (estimated from a subset of Weaver's data) used to arrive at a sample size may have been wrong or even meaningless. Second, in implementing the indirect scaling analysis method, the relative proportion of time a road is placed in a particular rating category by the subjects in the panel should be as similar as possible to the proportions that would be obtained from the population. This could only be ensured by using larger sample sizes. In addition, it was felt that the increased experimental control during the pilot study as well as its primary objective (i.e., to select a preferred rating method rather than to develop the final statistical relationship between objective and subjective data) reduced the requirement for a larger panel. A panel of 36 was ultimately used to derive the functional relationship between subjective ride quality and MRM data in the main study.

Panel Composition

Because balancing the composition of the panel by sex and age was the primary concern in selecting subjects, no explicit attempt was made to seek a balance based on other demographic variables. The panel was poststratified by miles driven per year, years of driving experience, vehicle size normally driven, seat position (front right, rear right, or rear left), scale used (one of three), and starting point (three starting points were used). The last three variables as well as the variable of sex were evenly divided among the panel.

DESIGN AND IMPLEMENTATION OF EXPERIMENT

Before they took the trip through the course, each group of three subjects was assigned to a particular combination of scale type and starting point (three alternative starting points were used to distribute the possible effects of learning and fatigue). The groups were then given their instructions.

Panel Instructions for Weaver-AASHO Scale

The panel instructions covered the purpose of the study, how the subjects were to use the rating scale, the procedures to be used in making their ratings, and other general topics. The instructions were outlined on a chalk board to ensure that all subjects received the same information each time. The instructions for the Weaver-AASHO scale are presented here. Instructions for the other scales are presented elsewhere (7).

> Highway Improvement Study
>
> Purpose: To survey typical Pennsylvania drivers in order to determine what they think of the quality of the ride provided by the roads in the Commonwealth. PennDOT will use this information to decide which roads to fix first given limited funds.
>
> Object of Study: To obtain your personal opinion of how rough or smooth a ride is provided by roads in the area which represent the condition of various roads throughout the Commonwealth.
>
> How to Make Your Ratings (A facsimile of the rating scale to be used was on the board for this section.)
>
> Object: To place a mark across the vertical line which you think best describes the ride provided by the roads.
>
> Definition of End Points
>
> Impassable: A road which is so bad that you doubt that you or the car will make it to the end at the speed you are traveling--like driving down railroad tracks along the ties.
>
> Perfect: So smooth that at the speed you are traveling you would hardly know the road was there. You doubt that if someone made the surface smoother the ride would be detectably nicer.
>
> All the roads which you drive over today will be between these two extremes. That is, since these roads probably do not exist you will probably not consider any road to be worse than impassable or better than perfect. In order to help you place your mark on the line, we have included a number of words along the scale which could be used to describe how the riding sensation seems to you.

For example, if you should encounter a road for which you could describe the ride as FAIR but not quite GOOD, place your mark just below the line labeled "3" (illustrated). On the other hand, if you think the next road is still fair, but somewhat worse than the previous road, place your mark at a point which you think is the appropriate distance down in the FAIR category. To indicate small differences between the ride quality provided by the roads, you may place your mark anywhere you like along the scale.

NOTE: We are not asking you to place roads into one of five categories! You should use small differences in the position of your marks to indicate small differences between the ride quality provided by the roads. You may place your mark anywhere you like along the scale.

Procedure We Will Use Today

1. We will drive over a predetermined course in an ordinary passenger car.
2. The trip will take about 3 hours depending on traffic conditions.
3. We will ask you to rate 33 road sections; you will not be rating an entire road.
4. It will only take about 30 seconds to drive over each section.
5. As you approach each site, the driver will call out the number of the site. Be sure you have the proper form.
6. When the driver says START, begin concentrating on what the rating should be based on how the ride feels to you.
7. Maintain your concentration until the driver says STOP.
8. At that point, place your mark on the scale and pass the forms to the person sitting in the front right seat.
9. Many sites are only 3-4 minutes apart, so make your ratings as quickly as you can.
10. This procedure will be repeated for each site.
11. There are planned rest stops but if anyone would like to stop sooner, tell the driver.

Special Instructions

1. Do not consider any of the road before or after a test section. We are only interested in a rating for a small section of road.
2. Concentrate only on the ride quality provided by the roads. Don't let the appearance of the road surface influence your ratings. Judge only how the road feels!
3. Don't be distracted by conversations in the car or by pretty scenery.
4. Don't reveal your ratings to the other raters. There are no right or wrong answers, so don't "cheat." We are interested only in your opinion which is as valid as anyone else's.
5. Be critical about the ride quality provided by the roads. If they are not absolutely perfect as far as you are concerned, be sure to give it a rating on the scale which you think best reflects the diminished quality of the ride.
6. Be aware that there are many ways that the ride could be considered less than PERFECT. The road could (a) be so bumpy that it rattles your bones and makes your teeth chatter, (b) have bumps or undulations which make the car heave up and down as if it were a boat in high seas, or (c) have other imperfections in the surface which you think detract from the ride quality.

Driving the Course

The trip to the beginning of the course always started with a drive through the parking lot of a nearby shopping center, which was generally considered to be representative of a very rough road. This served two purposes: it acted as a trial run through a site so that the subjects could practice making their ratings, and it demonstrated to the subjects how the test car (a 1981 Chevrolet Citation four-door sedan) performed on a rough road in comparison with their own vehicles. The subjects were aware of the purposes of this trial run but were not told what the rating for the parking lot should be; that is, the ride quality of the parking lot was not meant to define any particular point along the scale. All of the runs were completed in 13 (working) days.

ANALYSIS AND INTERPRETATION OF RESULTS

Data Reduction

The data were reduced by measuring, to the nearest 0.1 in., the distance between the raters' marks and the low end of the scale. Means and standard deviations were computed for each of the 33 sites for each of the three scales. In addition, the 13-step analysis procedure used by Weaver (4) was applied to derive indirect scale values. These data were transformed to provide all positive scale values. The data for the three scales are given in Tables 2 through 4.

Analysis of Variance by Rank

An analysis of variance by rank was conducted to determine which scaling method resulted in the great-

Table 2. Weaver-AASHO scale.

Site	Mean	Standard Deviation	Indirect Scale Value	Transformed Indirect Scale Value
1	1.81	0.85	-0.906	1.937
2	3.21	0.65	1.308	4.151
3	0.86	0.48	-2.367	0.476
4	1.29	0.57	-1.667	1.176
5	2.21	0.73	-0.134	2.709
6	2.62	0.71	0.538	3.381
7	3.33	0.54	1.552	4.395
8	3.89	0.57	2.355	5.198
9	2.82	0.85	0.759	3.602
10	3.84	0.63	2.233	5.076
11	3.04	0.73	1.015	3.858
12	3.82	0.50	2.145	4.988
13	3.51	0.66	1.811	4.654
14	3.59	0.69	1.892	4.735
15	1.42	0.46	1.433	4.276
16	3.53	0.53	1.766	4.609
17	1.42	0.46	-1.422	1.421
18	2.97	0.62	0.879	3.722
19	3.45	0.54	1.726	4.569
20	3.12	0.69	1.132	3.975
21	3.07	0.65	1.135	3.978
22	1.91	0.65	-0.611	2.232
23	3.14	0.66	1.264	4.107
24	0.67	0.37	-2.843	0
25	3.71	0.61	2.103	4.946
26	2.13	0.60	-0.301	2.542
27	1.39	0.65	-1.480	1.363
28	3.51	0.68	1.853	4.696
29	2.97	0.58	0.927	3.770
30	1.63	0.54	-1.101	1.742
31	2.21	0.49	-0.173	2.670
32	3.39	0.57	1.641	4.484
33	3.04	0.73	1.093	3.936

est agreement among the raters. This analysis yielded two statistics: a chi-square value and a coefficient of concordance. The chi-square value was used to test the null hypothesis that the subject yielded only random ratings on the test sections. As the data given in Table 5 indicate, the chi-square values for all three scales were quite significant because, as expected, the subjects easily detected a difference between at least two sections and rated them accordingly.

The coefficient of concordance is similar to a correlation coefficient and can be used to indicate which scaling method produces the greatest agreement among the subjects. The coefficient can range between zero and one; the closer it is to one, the better is the agreement between the subjects. Thus, this statistic can be used to discriminate between rating methods that produce significant chi-square values. The coefficients of concordance (and the related average intercorrelations) show that the three scaling methods are comparable to one another in producing reasonably good agreement among the subjects when they are rating sites. Although Holbrook's scale yielded the highest coefficient of concordance, it would be difficult to choose one method over another on the basis of these results.

Regression Analysis

Regression analyses were also conducted to get a preliminary indication of the form and strength of the relationship between the objective and subjective measures provided by each scaling method. Scatter diagrams of the mean ratings versus normalized axle displacements (inches per mile) are shown in Figures 2 through 4 and, in general, reveal linear relationships. Because a straight line accounts for 84 to 90 percent of the variance in these data, it was considered unlikely that another functional relationship could be found that would explain a significant amount of the remaining variance.

Table 3. Holbrook scale.

Site	Mean	Standard Deviation	Indirect Scale Value	Transformed Indirect Scale Value
1	2.37	0.65	0.132	2.576
2	3.93	0.43	2.415	4.859
3	1.06	0.40	-2.196	0.248
4	1.66	0.71	-1.008	1.436
5	2.70	0.69	0.542	2.986
6	3.26	0.57	1.250	3.694
7	3.89	0.50	2.421	4.865
8	4.45	0.25	3.275	5.719
9	3.06	0.67	1.184	3.628
10	4.34	0.41	3.290	5.734
11	3.79	0.50	2.187	4.631
12	4.44	0.35	3.331	5.775
13	4.16	0.58	2.917	5.561
14	4.08	0.54	2.707	5.151
15	4.17	0.46	2.958	5.402
16	3.91	0.54	2.379	4.823
17	1.41	0.53	-1.610	0.834
18	2.68	0.66	0.490	2.934
19	3.91	0.59	2.444	4.888
20	3.18	0.85	1.231	3.675
21	3.13	0.84	1.232	3.676
22	1.95	0.60	-0.727	1.717
23	3.56	0.75	1.926	4.370
24	0.85	0.45	-2.444	0
25	4.23	0.55	2.962	5.406
26	2.24	0.51	-0.085	2.529
27	1.53	0.57	-1.322	1.122
28	3.76	0.71	2.094	4.538
29	3.40	0.67	1.650	4.094
30	1.77	0.42	-0.966	1.478
31	2.35	0.64	0.046	2.490
32	3.92	0.53	2.528	4.972
33	3.70	0.65	2.124	4.568

Table 4. Nonsegmented scale.

Site	Mean	Standard Deviation	Indirect Scale Value	Transformed Indirect Scale Value
1	1.63	0.68	-1.057	1.868
2	3.47	0.79	1.606	4.531
3	0.74	0.46	-2.709	0.216
4	1.04	0.45	-2.125	0.800
5	1.84	0.75	-0.750	2.175
6	2.60	0.65	0.391	3.316
7	3.50	0.64	1.714	4.639
8	4.04	0.52	2.554	5.479
9	2.63	0.62	0.575	3.500
10	4.01	0.41	2.447	5.372
11	2.97	0.74	1.066	3.991
12	4.09	0.50	2.602	5.527
13	3.49	0.70	1.714	4.639
14	3.83	0.59	2.259	5.184
15	3.52	0.81	1.730	4.655
16	3.90	0.66	2.369	5.294
17	1.04	0.53	-2.091	0.834
18	2.73	0.68	0.615	3.540
19	3.51	0.54	1.826	4.751
20	3.33	0.77	1.459	4.384
21	3.11	0.65	1.089	4.014
22	1.82	0.79	-0.885	2.040
23	3.13	0.75	1.207	4.132
24	0.52	0.32	-2.925	0
25	4.23	0.41	3.049	5.974
26	2.31	0.76	-0.011	2.914
27	1.42	0.79	-1.547	1.378
28	3.76	0.74	2.136	5.061
29	2.78	0.62	0.653	3.578
30	1.53	0.40	-1.332	1.593
31	1.98	0.51	-0.519	2.406
32	3.29	0.68	1.395	4.320
33	3.23	1.01	1.386	4.311

Table 5. Results of analysis of variance by rank.

Scale	Chi-Square[a]	Degrees of Freedom	Coefficient of Concordance	Average Intercorrelation
Weaver-AASHO	439.95	32	0.764	0.750
Holbrook	461.61	32	0.801	0.790
Nonsegmented	427.66	32	0.742	0.727

[a] $P < 0.001$, crit $\chi^2(32) \approx 51$.

The linear relationship found between the objective and subjective measures was somewhat unexpected. Previous research (4) had suggested that the relationship between these two measures would tend to obey Fechner's law; that is, the subjective ratings (R) should be a logarithmic function of the physical stimuli (S), or

$$R = -a \log S \tag{1}$$

where a is a scale constant.

This formulation implies that small increases in road roughness for smoother roads should cause a greater decrease in the ratings than the same amount of change for rougher roads. In other words, people should be more sensitive to smaller differences in the variation of road roughness on very smooth roads than they are on rougher roads. One possible reason that this relationship was not found is that the high end of the function is not well defined. Only five sites produced an axle displacement of 550 in./mile or more. Perhaps if a larger number of sites with roughnesses in this range had been included in the study, the expected trend would have been found.

This issue aside, however, comparison of the three correlation coefficients shows that, as in the

Figure 2. Mean rating versus axle displacement for Weaver-AASHO scale.

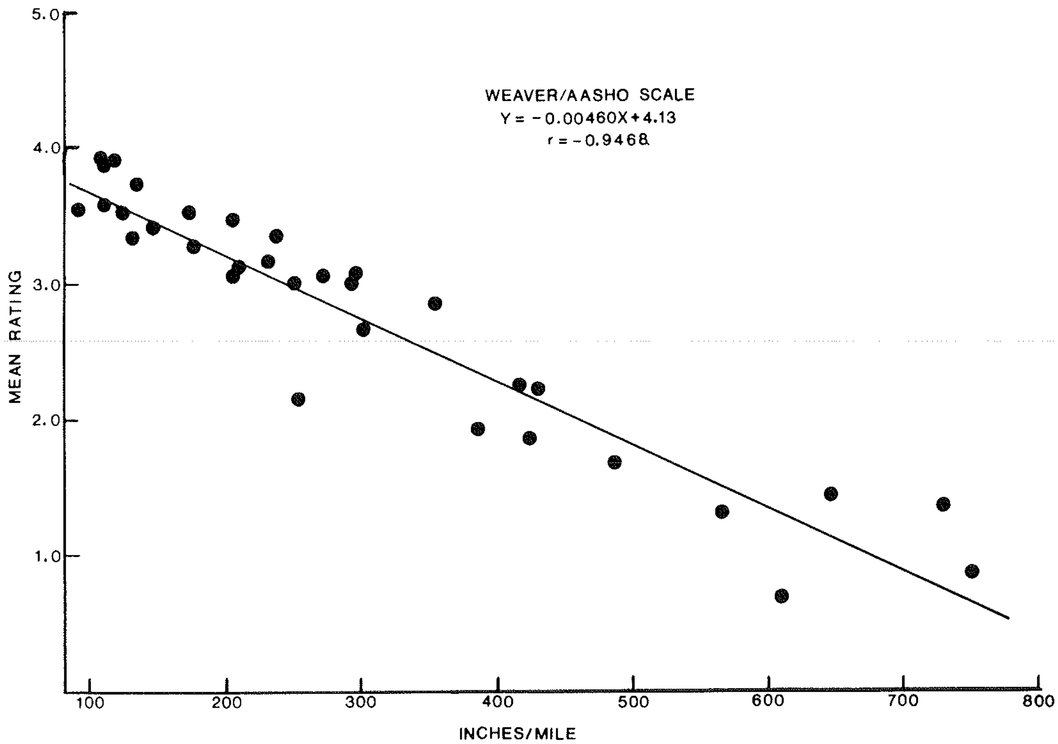
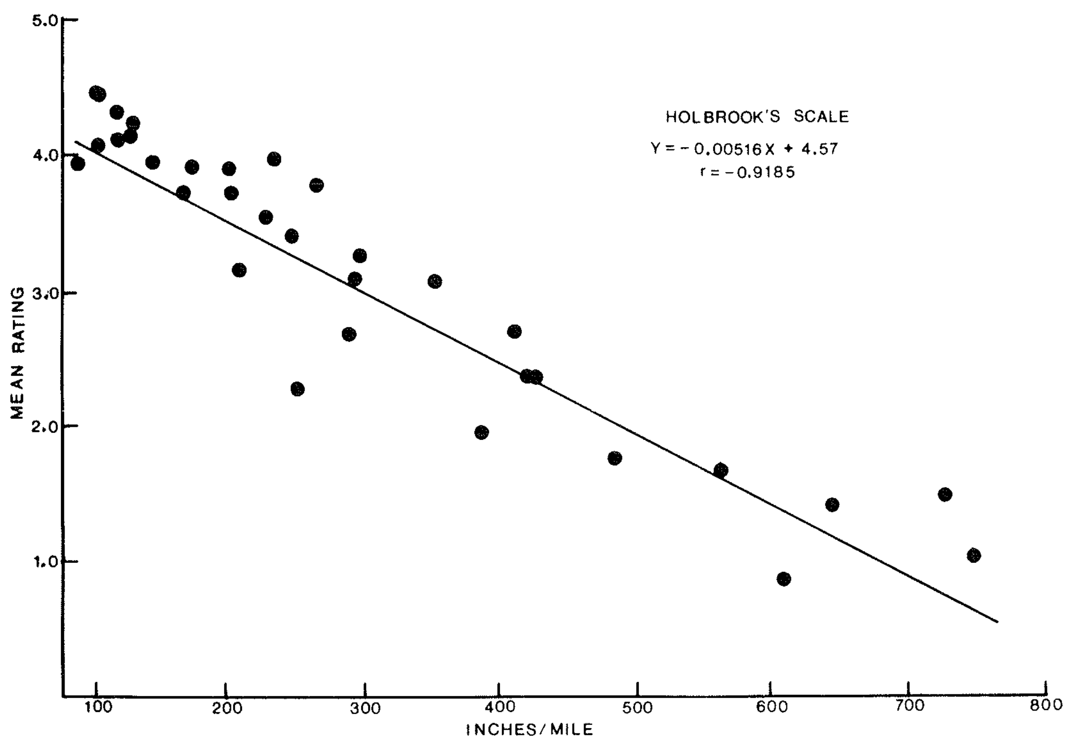
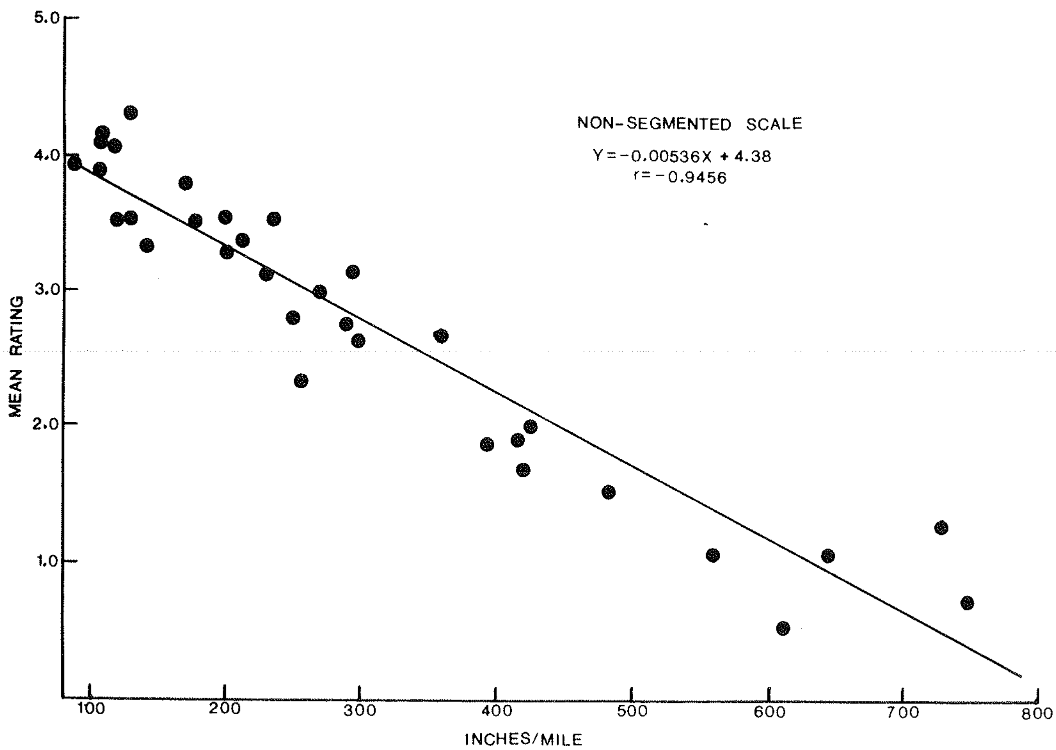


Figure 3. Mean rating versus axle displacement for Holbrook scale.



case of the results from the analysis of variance by rank, there is little reason for selecting one scaling method over the others on the basis of the correlations. The differences among the correlations can be attributed entirely to sampling variation. This conclusion is supported by the total overlap of the 95 percent confidence intervals (based on Fisher's Z-transform) for all three correlation coefficients. Therefore, there are no quantitative arguments for selecting one method over another. Consequently, for the sake of uniformity and the potential of comparing the data collected in this study with those collected over the years at NYSDOT, the Weaver-AASHO scale was selected for use in future work.

Figure 4. Mean rating versus axle displacement for nonsegmented scale.



Analysis of Direct Versus Indirect Scaling

When the transformed, indirect scale values (given in the last columns of Tables 2 through 4) were compared with the mean ratings (direct scale values), it was readily apparent that there was little difference between the two measures. A correlation of nearly 1.0 was found to exist between the two distributions (0.9981, 0.9986, and 0.9989 for the three scales). This suggests that the subjects are capable of making judgments directly on an interval scale (at least under these conditions). From a more practical standpoint, however, it appears that use of the method of successive categories is totally unnecessary in this situation. In a re-examination of Weaver's raw data, a correlation of 0.9917 was found between his direct scale values (i.e., averages) and indirect scale values (the method of successive categories), the same order of magnitude as those obtained in the research described here.

Other Analyses

The effects of starting point (learning and fatigue), seat position, sex, vehicle type normally driven, and average miles driven per year were found to be not significant. In addition, the results of a factor analysis showed that it was unlikely that the subjects rated the sites based on factors other than roughness.

One demographic variable that did prove to have a significant effect on ratings was the average number of years of driving experience (number of years licensed). Analysis showed that inexperienced drivers hesitated to give extreme ratings and were somewhat uncertain when they did rate the roads whereas more experienced drivers tended to show less variability and a willingness to give more extreme ratings. The implication of this is that care should be taken to avoid overrepresentation of inexperienced drivers on future panels.

CONCLUSIONS

The overall conclusion drawn from this phase of the study is that, if sites and panel members are carefully selected, subjects are properly instructed, and extraneous variables are controlled, quite similar results will be obtained with any of the scaling methods evaluated in this study. Thus, the choice of a scaling method can be and should be made on the basis of some attribute or attributes other than the ability of the subjects to use the scale, the correlation of the scale values with physical measures of road roughness, and other such factors.

Furthermore, no evidence was found to support the notion that the original method used by Carey and Irick violated good psychometric principles. The similarity of results among scales intended to remove systematic errors and the Weaver-AASHO scale appears to indicate that these errors do not play a significant role in the rating of ride quality (at least in a well-designed experiment). In addition, the strong correlation between direct scale values (averages) and indirectly obtained scale values also suggests that subjects are capable of making judgments directly at an interval level of measurement as Carey and Irick had implicitly assumed.

ACKNOWLEDGMENT

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REFERENCES

1. L.F. Holbrook. Prediction of Subjective Response to Road Roughness by Use of the Rapid Travel Profilometer. HRB, Highway Research Record 291, 1969, pp. 212-226.

2. W.N. Carey and P.E. Irick. The Pavement Serviceability Performance Concept. HRB, Bull. 250, 1960.
3. B.G. Hutchinson. Principles of Subjective Rating Scale Construction. HRB, Highway Research Record 46, 1963, pp. 60-70.
4. R.J. Weaver and R.O. Clark. Psychophysical Pavement Serviceability. Soil Mechanics Bureau, New York State Department of Transportation, Albany, May 1977.
5. J.P. Guilford. Psychometric Methods, 2nd ed. McGraw-Hill, New York, 1954.
6. W.S. Torgerson. Theory and Methods of Scaling. Wiley, New York, 1967.
7. J.B. Nick and M.S. Janoff. A Comparative Study of Psychophysical Scaling Methods for Evaluating Pavement Ride Quality. Pennsylvania Department of Transportation, Harrisburg, draft, Dec. 1981.

High-Speed Road Monitoring System

P.G. JORDAN AND J. PORTER

A high-speed road monitoring system has been developed at the Transport and Road Research Laboratory. It consists of four laser sensors mounted on a 4.5-m-long beam that is supported by a two-wheeled trailer towed behind a small van. Measurements are made by the configuration of laser sensors under the control of a computer system located in the vehicle behind which the trailer is towed. Longitudinal profile, wheel-track rutting, and surface macrotexture are measured as the system travels over the road networks in the normal traffic stream; provision is being made for the measurement of road crossfall, gradient, and horizontal curvature. The principles of system operation in the different measurement modes are described and illustrated. Use of the measurements made by the high-speed system in studies of the effects of unevenness on the road user and in detecting structural deterioration of roads is described. Its potential for use in making surveys of the road network at a relatively low cost, locating areas of distress, and guiding the deployment of other, more specialized equipment is discussed within the context of the development of a cost-effective maintenance management system.

Large sums of money are being spent throughout the world on road maintenance. Effective use of these funds demands careful allocation of resources. In the United Kingdom the Report of the Committee on Highway Maintenance (1) recommended that highway authorities should use an objective maintenance rating system as the basis of regular road inspections. In answer to this recommendation, various computerized highway maintenance systems (2) have been developed and are in widespread use in the United Kingdom. All rely heavily on visual inspection to assess the condition of the road surface by reference to, for example, wheel-track rutting and surface cracking. These visual condition surveys are increasingly being augmented by input from machines that monitor various aspects of road condition more quickly than could a team of inspectors.

A high-speed monitoring system that measures a number of different aspects of road condition has recently been developed at the Transport and Road Research Laboratory (TRRL). Measurements are made by the system as it travels over the network in the normal traffic stream. The power of the equipment lies in its ability to cover a large distance each day, gather surface profile and alignment data that describe the condition of the network, and guide the deployment of other slower, more specialized evaluation equipment.

In this paper, the road monitoring system is described and its use in research on the effects of surface unevenness and its development as a component part of a total maintenance management system are discussed.

DESCRIPTION OF THE SYSTEM AND ITS USE

The high-speed road monitoring system (3), shown in Figure 1, is a laser-based system that accurately measures road surface characteristics. Its on-board computer facilities, shown schematically in Figure 2, provide both measurement control and an on-site data-processing capability. The system operates at speeds between 5 and 80 km/h, and its performance is not affected by variations in speed. With a driver and one operator, it can cover as much as 200 km of profile, rutting, texture, and road alignment parameters each day. Data are stored on floppy disks and can be either processed on site or transferred to a central mainframe computer for permanent storage and further processing.

Figure 1. High-speed road monitoring system.

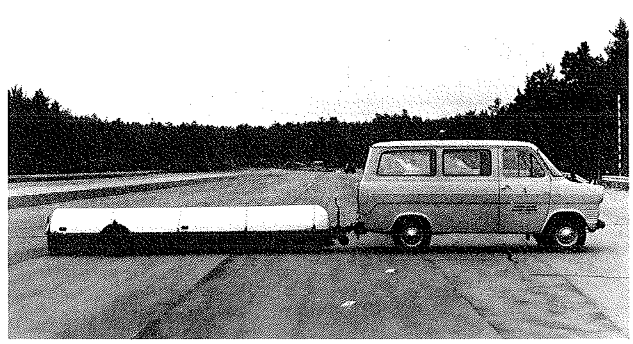
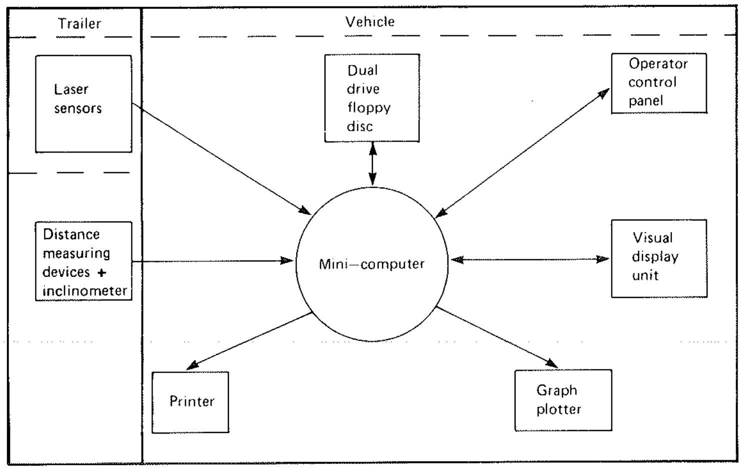


Figure 2. Computer control and analysis facilities of high-speed road monitoring system.



Laser Sensor

The essential features of the laser sensor (4) are shown in Figure 3. Laser light projection and receiving units are fixed to a rigid beam in such a way that the projection and receiving axes are orthogonal. The semiconductor laser source produces infrared light at pulse rates up to 3000/sec. The lens in the projection unit focuses the light from the laser on a 0.3-mm^2 spot on the road surface. This light is diffusely scattered by the surface; some is collected by the lens in the receiving unit and focused onto a linear array of photodiodes.

When the surface is displaced vertically in relation to the sensor, the illuminated spot moves along the axis of the projection beam. Because the projection and receiving axes are orthogonal, the image of the illuminated area formed on the diode array moves along the array and remains in focus, as shown in Figure 3. By detecting which diode in the array has maximum light intensity, the vertical distance of the sensor from the surface can be calculated.

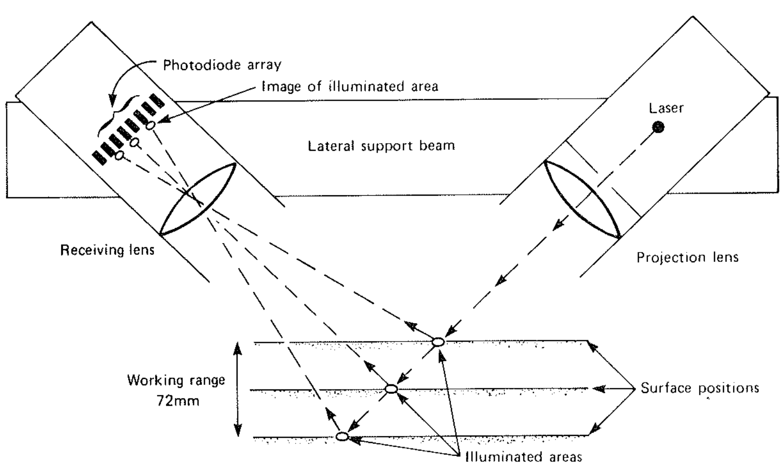
The sensor has a working range of 72 mm centered at a point 270 mm from the sensor, and resolution is +0.282 mm over the full working range.

Profile Measurement

For profile measurement the high-speed system (5) uses four laser sensors fitted to a 4.5-m-long rigid beam in the configuration shown in Figure 4. The sensor configuration provides two profile measuring systems: the symmetric (SYM) and the asymmetric (ASY). Each measures a different range of wavelengths. Profile features with wavelengths greater than 10 m are measured by using the SYM system (sensors A, C, and D in Figure 4); this computes the average profile height over a 2.14-m length (every 2.14 m) to give the long-wavelength profile of the road. The ASY system (sensors A, B, and D in Figure 4) is used to determine the profile in the 0.3- to 20-m wavelength range in terms of the average profile height over a 0.107-m length (every 0.107 m along the road). A single composite profile containing the wavelengths measured by both systems is obtained by superimposing the ASY profile on that measured by the SYM system.

The principle of operation of the profilometer is described in detail elsewhere (5). Its main features can be briefly illustrated by referring to the SYM measuring system. By moving the equipment forward a distance equal to that between the SYM sensors, the average heights h_A, h_C, and h_D of each of the sensors A, C, and D above the road surface can be computed. The averaged profile height (Y_1) of the road traversed by sensor D is referred to a datum line defined by joining the averaged heights at the center points of the lengths traversed by sensors A and C to give the averaged characteristic measurement (U_1) of the SYM system:

Figure 3. Essential features of laser sensor.



$$Y_1 = U_1 = -(h_A - 2h_C + h_D) \quad (1)$$

An important feature of Equation 1 is that the characteristic measurement U_1 is, for all practical purposes, unaffected by changes in height or pitching of the equipment provided the surface remains within the working range of the sensors.

The equipment is now moved forward so that the positions along the surface of the averaged profile heights measured by sensors C and D are coincident with the positions of previous measurements of sensors A and C. The new averaged profile height (Y_2) is calculated by using the averaged characteristic measurements U_1 and U_2 as follows:

$$Y_2 = 2U_1 + U_2 \quad (2)$$

In general, after n steps the averaged profile height (Y_n) is given by

$$Y_n = \sum_{i=1}^{n} (n - i + 1) U_i \quad (3)$$

In the ASY system the spacing of sensors A and B in Figure 4 is 0.05 that of the SYM sensors and ASY measurements are obtained at this spacing. By using a derivation similar to that given previously, it can be shown that after m steps the profile height (Y_m) of the ASY system averaged over the ASY step length is given by

$$Y_m = [40\, Y_{(m-1)} - Y_{(m-40)} + 39\, W_m]/39 \quad (4)$$

where W is the averaged characteristic measurement of the ASY system, which in terms of the averaged sensor measurements (h_A, h_B, and h_D) is given by

$$W = -(39\, h_A - 40\, h_B + h_D)/39 \quad (5)$$

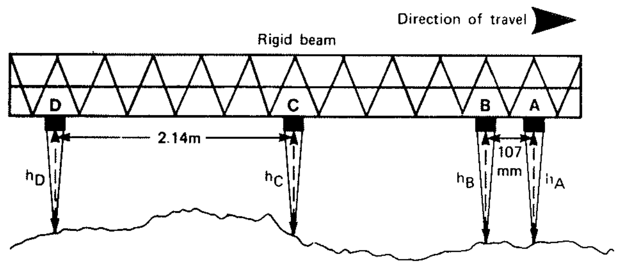
Equation 3 defines the long-wavelength features of the profile and Equation 4 defines the short-wavelength features. A composite profile containing both the long- and short-wave features is obtained by superimposing the ASY measurements on the SYM measurements by using a piecewise linear fitting process that is described in detail elsewhere (5). The low center of gravity of the trailer combined with the averaging used to reduce the effects of texture ensure that roll has a negligible effect on the measurements.

The performance of the TRRL high-speed system has been extensively investigated, not only by TRRL (5) but also independently by Bundesanstalt fur Strassenwessen (BAST) of West Germany (6). TRRL made detailed rod-and-level surveys at 0.1-m spacing, and BAST made continuous measurements with their profilograph along a defined line previously measured by the high-speed equipment. Bituminous and concrete surfacings containing different levels of short- and long-wave unevenness were measured. When the survey and high-speed measurements were correlated, coefficient values ranged from 0.96 to 0.99 for the surfacings examined and for profile wavelengths up to 50 m. For wavelengths up to 100 m the correlation coefficient is 0.95.

Figure 5 shows the system to have good amplitude and phase response between 0.5 and 100 m; good response is obtained up to wavelengths of several hundred meters, the upper limit being determined mainly by the level of macrotexture on the road surface (5). For wavelengths shorter than 0.5 m the amplitude response decreases because of the averaging involved in the derivation of the ASY measurements.

Accuracy of measurement is affected by road surface and machine factors; the maximum tolerances expected for the measurement of amplitudes at wavelengths up to 100 m are shown in Figure 5. The ability of the system to measure long wavelengths accurately means that it can be used to examine not only the riding quality of highways and airfields but also subsidence problems. An example of subsidence measured over an 800-m length of concrete pavement is shown in Figure 6. Comparison with conventional survey measurements on this site showed agreement to be within 10 percent at the maximum amplitude.

Figure 4. Sensor configuration on rigid beam for profile measurement.



Measurement of Wheel-Track Rutting

The method used for the measurement of wheel-track rutting (7) is shown in Figure 7. The wheels of the trailer ride in the ruts, and the laser sensor continuously measures the axle displacement from the road surface along a line centered between the wheel tracks. The difference between the axle displacement measured on a rutted surface and the data obtained on a nonrutted surface gives the rut depth averaged over both wheel tracks as shown in Figure 7.

To smooth the effect of the oscillatory motion of the trailer on its suspension, the rut measurements are averaged over a length directly proportional to the speed of the measuring system: e.g., for an operating speed of 50 km/h the length is 20 m.

Comparisons between measurements obtained by using the laser system and average rut depths derived from straightedge and wedge measurements show agreement to within 2 mm on major roads and 3 mm on minor roads (the difference arises from the greater camber on minor roads) (7).

Examples of rut depth profiles obtained by using the laser system at normal traffic speed on a motorway and on a single-carriageway principal road are compared in Figure 8 with rutting levels based on CHART system recommendations (2). The motorway is shown to have an acceptable level of rutting, but the principal road has sections that would require further investigation.

The computer facilities, shown in Figure 2, enable the storage of as much as 500 km of rut data per floppy disk. Lengths of road with critical levels of rutting can be located quickly by using the on-board analysis programs.

Measurement of Macrotexture

Macrotexture is measured (8) by using sensor C of the high-speed road monitoring system (Figure 4), which measures about 3,000 displacements/sec as it moves over the surface. The macrotexture appears in the form of small random variations in the measure-

Figure 5. Amplitude and phase-response measurement tolerances of amplitude.

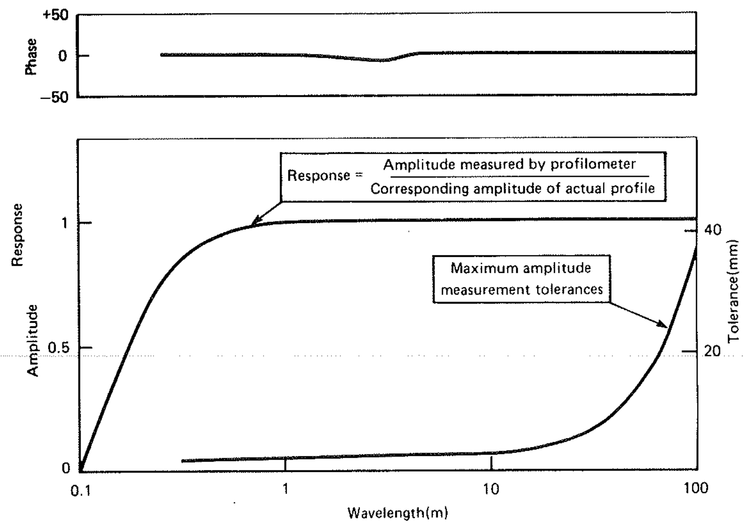
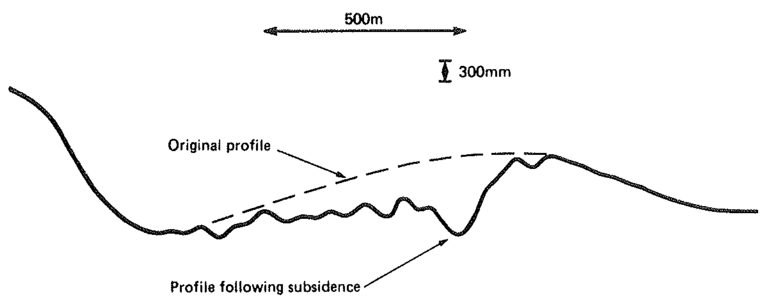


Figure 6. Measurement of subsidence profile on a continuously reinforced concrete pavement by the high-speed road monitoring system.



ments superimposed on the much larger oscillatory motion induced by the trailer suspension (see Figure 9). To obtain a measure of the macrotexture depth (8), the oscillatory motion is characterized by a piecewise parabolic curve-fitting procedure. The texture depth is then calculated in terms of the standard deviation of the residual displacements, as shown in Figure 9. The standard deviation of texture depth is calculated over short lengths of surface, usually 0.3 m; it can be presented in the form of a histogram or profile of texture depth along the road surface.

Comparison of texture measurements made by the laser sensor with those made by the more traditional sand-patch method (3) show good agreement: correlation coefficients exceed 0.9 (8). Selected sections of motorway ranging from 6 to 21 km long have been measured by using the laser system. The results have shown that the system is capable of continuous measurement of macrotexture at operating speeds up to 35 km/h (9). On grooved concrete surfaces the laser system has also been successfully applied to the measurement of groove depth and spacing (10).

Measurement of Road Cross Slope and Radius of Curvature

By using essentially the system shown in Figure 2 but with the addition of transverse and longitudinally positioned inclinometers, the capability to measure road gradient, crossfall, and radius of curvature of bends is being incorporated into the high-speed road monitoring system. The radius of curvature of bends is computed by using the difference in the distances traveled by the wheels of the trailer on rounding a bend. Road gradient and crossfall are derived from inclinometer measurements. The effects of radial acceleration on crossfall measurements are corrected by using the measured radius of curvature. Trailer roll effects are reduced to an acceptable level by averaging the crossfall measurements over a length that is related to operating speed; typically, a length of 20 m is used for a speed of 50 km/h.

Preliminary test trials have shown system measurements to be in good agreement with those obtained by using conventional survey methods.

RESEARCH ON EFFECTS OF ROAD SURFACE UNEVENNESS

To exploit fully the capabilities of the road monitoring system, more must be known about the effect of surface unevenness on (a) the comfort and safety of the road user, (b) vehicle operating costs, (c) damage to the road structure, and (d) occupants of buildings adjacent to busy roads. At TRRL the various aspects of this problem are being investigated to determine the consequences of surface deterioration and, ultimately, define intervention levels for maintenance purposes.

Effect on Road Users

As a result of extensive studies (11,12) ride assessment by vehicle occupants has been related to surface unevenness. In these studies vehicle vibra-

Figure 7. Measurement of wheel-track rutting with high-speed road monitoring system.

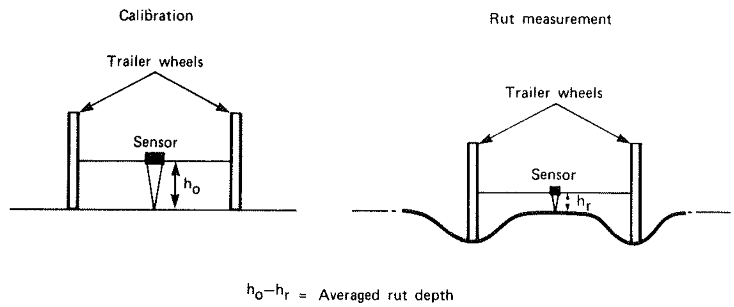


Figure 8. Variation of wheel-track rutting along a motorway and a principal road.

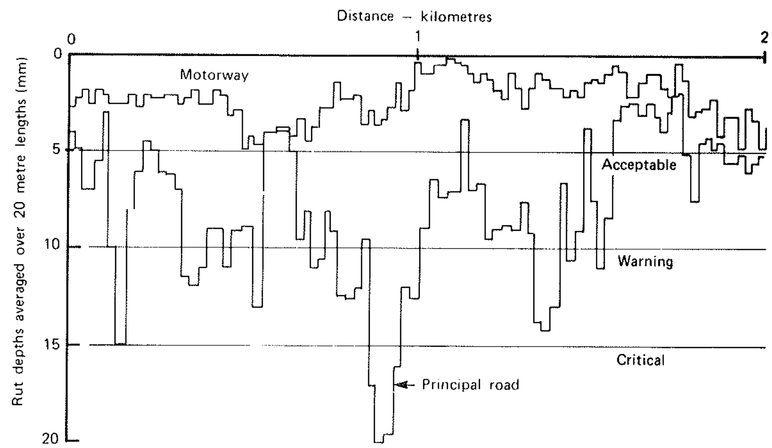
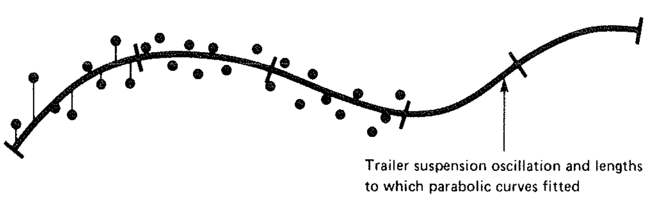


Figure 9. Measurement of macrotexture with laser sensor system.



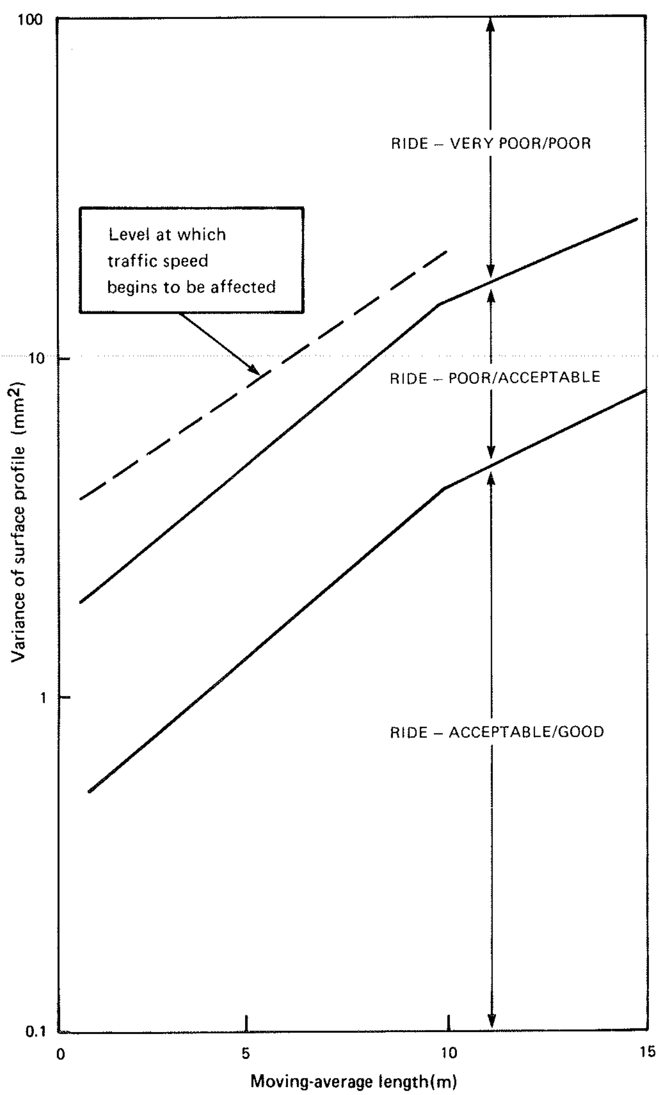
tion, defined by the root mean square (RMS) of vertical acceleration, was correlated with subjective assessments of ride by both a panel of expert assessors and large samples of the motoring public driving their own vehicles over selected sites. The test sites had a range of unevenness and carried traffic operating at speeds greater than 70 km/h. Because of the inherent variability of the subjective assessments, the results of the studies have been interpreted in probabilistic terms. Relations have been computed that give the probability of a ride being rated acceptable or better for given levels of RMS acceleration. For RMS accelerations of less than 0.04 g the analyses show that 90 percent of motorists in automobiles rated the ride as acceptable or better. Motorists driving their own cars were found to be less critical of ride than the panel of expert assessors and truck drivers, and bus passengers were more tolerant of vibration than automobile passengers.

By using profiles of the test sites measured by the road monitoring system, the RMS acceleration values have been correlated with unevenness to give ride criteria for new and in-service roads. Figure 10 shows evenness criteria for roads with traffic speeds greater than 70 km/h; unevenness is defined in Figure 10 in terms of the variance of profile deviations from a moving-average datum. Each variance value reflects the profile unevenness that is associated with profile features less than or equal to the length of the moving average. In the practical application of these criteria, profiles measured by the road monitoring system would be analyzed by using the on-board computer to give the variance values associated with moving-average lengths of 3, 7, 10, and 15 m for comparison with the criteria and thereby define the riding quality of the profile. This analysis is made based on profile lengths selected by the user, typically 300 m in the United Kingdom.

The effect of unevenness on traffic speed has been investigated as part of a general study of the consequences of surface deterioration (13). The level of unevenness at which traffic speed began to be affected on major roads in the United Kingdom is shown in Figure 10. It was concluded that, under existing maintenance practice, deterioration of the surface profile was unlikely to cause a decrease in traffic speed of more than 3 km/h on major roads. Because there are relatively few major roads in the United Kingdom on which unevenness exceeds the level shown in Figure 10, the benefit to road users, in relation to reduced travel times, from improving the profiles on major roads is marginal.

Other consequences that are being investigated include vehicle maintenance and fuel consumption. A pilot survey of vehicle fleet operators indicated that vehicle maintenance was not affected by the levels of unevenness on major roads in the United Kingdom. On other roads there was some indication of an increase in vehicle maintenance that was ascribed to unevenness.

Figure 10. Ride (evenness) criteria for use on major in-service roads.



The effects of unevenness on energy dissipation through tire damping and suspension losses and, hence, fuel consumption are being examined. Initial results show that there is a small but significant increase in energy dissipation for vehicles operating on roads that have levels of unevenness representative of those found on some urban and minor rural roads; the level of energy dissipation depends on vehicle speed and on the spectrum of surface unevenness.

Skidding and Safety

The safety of the road user is a primary consideration of the maintenance engineer in evaluating the condition of in-service roads. Road safety is a complex issue involving the road user, the vehicle, climate, and road conditions. Although it is difficult to quantify the contribution of these elements to road safety, it has been estimated that road-related factors are involved in about a quarter of all road vehicle accidents (14). Skid resistance, particularly in wet conditions, has been shown to have a significant effect on accident risk (14). Aggregate characteristics, surface texture, and, to a lesser extent, rutting are directly related to skid resistance (15), but their effect in any given situation is also greatly affected by road alignment. By using the data provided by the high-speed road monitor together with skid resistance data, the effect of road alignment and road surface parameters (both individually and in combination) on accident risk can be examined by means of correlation methods. Once the parameter levels associated with a significant accident risk were determined from studies, the equipment would be used to survey road alignment and surface condition to locate those sections of the network that have a significant accident risk. With a knowledge of traffic densities and speeds, the cost in human and material resources of road-related safety measures can be better estimated.

Structural Deterioration

The traditional method of detecting structural deterioration on roads in the United Kingdom is visual inspection. The development of the deflectograph (16) and of criteria for interpreting its measurements (17) has resulted in widespread use of this instrument as a means of providing an objective evaluation of structural conditions in quantitative terms. A disadvantage of both visual inspection and the deflectograph for survey work is their relative slowness.

The high-speed road monitor has been used to carry out a continuing survey of longitudinal profile on a sample of different road constructions over a period of 4 years. Analysis of the data from this survey shows that the rate of change of unevenness correlates better with the structural condition of pavements than does the absolute level of unevenness; the rate of change of unevenness, defined as the proportional change in profile variance over 10 percent of the nominal life of the test site, was computed for each 25-m length of each test site. The profile variance was computed in relation to a datum derived from a 3-m moving average of the profile.

Work is continuing on the development of criteria for the early detection of structural distress on major roads. In practice, those sections where structural distress was indicated would then be surveyed in detail by using the deflectograph and visual inspection to ascertain the nature and extent of the distress and determine the necessary strengthening treatments (17).

PAVEMENT MAINTENANCE OPTIMIZATION

With about £10,000/km being spent each year on maintenance of the U.K. trunk road system and more than £1,000 on other roads [£1 = U.S. $2.32 (1980)] there is clearly scope for investing money in techniques and equipment that help to reduce the total annual maintenance bill and allocate the available funds in the most cost-effective manner.

In response to economic pressures maintenance management has developed slowly over the years. As new equipment and techniques have become available they have been put to effective use, but without producing a complete solution to the overall problem of allocating resources. The techniques used include engineering judgment; visual inspection in surveys such as CHART, which cost between £30/km (rural) and £250/km (urban); SCRIM, which costs £10/km; and the deflectograph, which costs £90/km. With the advent of the high-speed road monitor, which can survey the network at up to 200 km/day at a cost of about £5/lane-km, and parallel improvement of the bump integrator (18), two powerful survey tools have been made available that can take the subjective judgment out of the first stage of inspecting the network.

For the development of a satisfactory economic model of maintenance management, the various measures of pavement condition should be linked to costs of remedial treatment, vehicle operating cost, and non-road-user costs. The results of road condition surveys might then be used to adjust maintenance interventions to minimize the total discounted cost to the community.

In a road investment model developed by the Overseas Unit of TRRL for use in countries with unpaved roads, an economic optimum is more closely approached by taking into account the effect of highway maintenance standards on vehicle operating costs (19). Vehicle operating costs are, of course, much less affected by the relatively lower levels of deterioration of the generally better-quality paved surfaces in developed countries. However, the accuracy of measurement and the processing capability of the high-speed road monitor allied with studies on the effects of unevenness such as those described previously now provide a means of examining this difficult problem.

CONCLUSIONS

Described in this paper is the operation of a high-speed road monitoring system developed at TRRL that measures longitudinal profile, rut depth, and surface texture. Work is underway to include crossfall, gradient, and horizontal curvature. As further enhancements are required they can be added to the system to make it even more effective for rapidly surveying the condition of the road network.

Longitudinal profile and rut measurements provide an assessment of ride quality and indicate structural condition. Although the high-speed road monitor does not measure skid resistance directly, its measurements of road alignment, profile, surface texture, and rutting can be used to locate potentially hazardous sections of the road network.

The contribution the high-speed road monitor can make to a maintenance management system has been briefly discussed, particularly its ability to provide up-to-date information on the condition of the network and to direct more specialized equipment to those distressed areas where its use would be most cost effective.

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REFERENCES

1. U.K. Ministry of Transport. Report of the Committee on Highway Maintenance. Her Majesty's Stationary Office, London, 1970.
2. P.J.F. Wingate and C.H. Peters. The CHART System of Assessing Structural Maintenance Needs of Highways. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 153 UC, 1973.
3. P.G. Jordan. Measurement of Road Surface Shape Using a Laser Sensor. In Sensors in Highway and Civil Engineering, Institution of Civil Engineers, London, 1981.
4. P.B. Still and M.A. Winnett. Development of Contactless Displacement Transducer. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 690, 1975.
5. P.B. Still and P.G. Jordan. Evaluation of the TRRL High-Speed Profilometer. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 922, 1980.
6. H. Braun and F.-J. Laermann. Comparative Measurements of Unevenness Between the TRRL High-Speed Profilometer and the Braunschweig-Profilograf. Strasse und Autobahn, No. 12, 1980.
7. P.G. Jordan and P.B. Still. Measurement of Rut Depths in Road Surfaces by the TRRL High-Speed Profilometer. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 1037, 1982.
8. D.R.C. Cooper. Measurement of Road Surface Texture by a Contactless Sensor. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 639, 1974.
9. J.R. Hosking and P.B. Still. Measurement of the Macrotexture of Roads: Part 1--Trials with the Contactless Sensor on Motorway M1. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 498, 1979.
10. P.B. Still. Measurement of Groove Dimensions on Concrete Pavements by a Contactless Sensor. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 443, 1978.
11. D.R.C. Cooper and J.C. Young. Road Surface Irregularity and Vehicle Ride. In Human Factors in Transport Research, Academic Press, London, Vol. 2, 1980, pp. 105-113.
12. J.C. Young and P.G. Jordan. Road Surfacings and Noise Inside Saloon Cars. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 655, 1981.
13. D.R.C. Cooper, P.G. Jordan, and J.C. Young. The Effect on Traffic Speeds of Resurfacing a Road. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 571, 1980.
14. B.E. Sabey. Accidents: Their Cost and Relation to Surface Characteristics. Presented at Symposium on Safety and the Concrete Road Surface: Design, Specification, and Construction, Birmingham, England, 1973.
15. G.F. Salt. Research on Skid Resistance at the Transport and Road Research Laboratory (1927-1977). Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. SR 340, 1977.
16. C.K. Kennedy, P. Fevre, and C.S. Clarke. Pavement Deflection: Equipment for Measurement in the United Kingdom. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 834, 1978.
17. C.K. Kennedy and N.W. Lister. Prediction of Pavement Performance and the Design of Overlays. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 833, 1978.

18. P.G. Jordan and J.C. Young. Developments in the Calibration and Use of the Bump Integrater. Journal of the Institute of Highway Engineers, No. 4, April 1981.
19. R. Robinson, et al. A Road Transport Investment Model for Developing Countries. Department of the Environment, Department of Transport, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 674, 1975.

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Use of Response-Type Roughness Meters for Pavement Smoothness Acceptance in Georgia

WOUTER GULDEN, JERRY STONE, AND DENNIS RICHARDSON

The use of response-type, road-roughness-measuring systems as part of surface tolerance specifications is attracting increasing interest among highway agencies as a rapid, inexpensive means of measuring the smoothness of roads during and after construction. Problems such as calibration, vehicle maintenance, and the repeatability of test results must be taken into account and resolved or minimized when these roughness-measuring systems are used for acceptance or rejection of projects for smoothness. The Georgia Department of Transportation has been using road meters in its specifications since 1972 for acceptance of projects and since 1979 for both rejection and acceptance. The evolution of the road-roughness-measuring program in Georgia, the calibration and operating procedures, the current smoothness specifications, and the use of Mays meter data during construction are described.

The surface smoothness testing program of the Georgia Department of Transportation (DOT) has evolved over the years from the rolling straightedge to trailer-mounted Mays ride meters and from testing for information purposes only to project construction control and acceptance. Many changes in equipment and procedures were made during this evolution to enhance the program and to ensure acceptance of the test results by contractors and project engineers alike.

Before 1966 the rolling straightedge was used to measure and control pavement roughness. Realizing the shortcomings of the straightedge in relating surface profile deviations to ridability, the Georgia DOT began to experiment with the CHLOE profilometer, but it soon became obvious that this device was too slow to be used in a large-scale program.

In 1968 Georgia began using the Portland Cement Association (PCA) road meter on a limited basis to check the roughness of various Interstate projects and some other selected paving projects. The road meter was installed in a carry-all type of vehicle, although it was designed to be installed in a standard-sized car.

The road-meter program was expanded in 1972 with the purchase of a PCA meter for each of the seven highway districts in Georgia so that each paving project could be monitored during construction. Each project was also measured for ridability before construction so that it would be possible to determine the amount of improvement in ridability after resurfacing.

The test results had previously been provided to contractors and project engineers so that they could become familiar with roughness testing and the results that were being obtained. In 1972 the Georgia DOT began using the PCA meter in lieu of the straightedge for acceptance of pavement smoothness on construction projects. If a project met the PCA meter specification it was accepted without further testing, but if it failed to meet these ridability requirements the failing sections were then retested with the rolling straightedge. The PCA meter was therefore used only as an acceptance tool, and any penalties were assessed based on rolling-straightedge results.

The carry-all vehicles were replaced during the next few years, and each district purchased replacement vehicles independently. By 1976 the meters were mounted in a variety of vehicle types, such as suburbans, station wagons, and cars of various sizes and makes. During this time the PCA meter was still used for acceptance testing only, and variations in road profile response from the various vehicles were unimportant to the contractor because penalties were still being assessed based on straightedge results.

Monitoring of the results obtained with the PCA meter and the rolling straightedge showed no consistent correlation between these two devices. Frequently, a section that failed the PCA meter requirements would be assessed no penalties based on the rolling-straightedge method. Sections determined to be acceptable by the road meter were sometimes found to have failed the straightedge requirements. It was obvious that the two devices did not give compatible results on all types of roads and roughness levels.

In 1975 the decision was made to standardize the PCA meter so that it could eventually be used for construction control and entirely replace the rolling straightedge. A testing program was conducted to compare PCA meter results obtained by various vehicles. The station wagon was chosen as a standard test vehicle and a fleet of station wagons was purchased.

Several other changes were made at the same time in an effort to standardize the equipment and upgrade the reliability of the testing program. An automatic null system was added to all PCA meters, radial tires were used on all test vehicles, and tires were trued and balanced as necessary. A lever arm system was developed so that individual units could be mechanically adjusted to produce comparable readings. A precise speed-deviation meter was used so that testing speed could be maintained to ±1 mph. A spare bank of counters was added to allow for continuous testing, and cruise control was used to maintain a uniform testing speed.

In addition, an operator's manual was written and all operators were given a training course in the use of the PCA meter. The manual contained detailed operating instructions; standard specifications for factors such as speed, tire pressure, and shocks; and detailed procedures for maintaining calibration. Calibration test sections were also established for each district.

By 1979 the roughness-testing program had progressed to the point where the PCA meter was used for construction control. Pavement sections that did not meet the roughness specifications after construction or resurfacing had to be corrected by the contractor at no cost to the state DOT.

By this time it was becoming apparent that it would be difficult to maintain a standardized vehicle over a period of years. Problems occurred when the road-meter vehicles had to be replaced because of excessive mileage and wear and tear on the suspension system. The response of the vehicle to road roughness is related to vehicle suspension and damping characteristics. It is important, therefore, that replacement vehicles have the same basic weight, wheelbase, and suspension characteristics as the original vehicle.

Vehicles that are currently being manufactured are increasingly smaller and lighter in weight. This means that existing standards for defining the roughness of a road would be altered every time the road-meter vehicles needed replacing. To eliminate this problem it was decided to mount the roughness-measuring equipment in a standardized test trailer.

A drawback of the PCA meter system was that it provided the roughness level for a section but could not distinguish where in a section the roughness was located. Such data are important because the specifications require correction of failing sections and each section tested is normally 1 mile long. The PCA meter does not indicate whether the entire section is rough or excessive roughness comes from specific areas within the tested section. Therefore, it was decided to change the testing equipment from a vehicle-mounted PCA meter to a trailer-mounted Mays meter.

In 1979 the first trailer-mounted Mays meter was acquired for experimental purposes. Based on the favorable results obtained with the first unit, trailers were purchased for each district in 1980 and the Mays meter system was put into full operation on January 1, 1981. During the transition period, projects contracted that used the PCA meter specifications were accepted based on Mays meter results. If a section failed, retests were conducted with the PCA meter in order to be fair to the contractor. All projects that were contracted after January 1, 1981, were tested with the Mays meter only.

PRESENT ROUGHNESS-TESTING PROGRAM AND EQUIPMENT

The trailer-mounted Mays meter is used in each highway district in Georgia to measure the pavement roughness of all projects before, during, and after construction. Each district is responsible for scheduling, making roughness measurements, and calibrating the trailers a minimum of every 2 weeks. The central office has overall responsibility for the program, monitors the results of the district calibrations, trains new operators, determines test procedures and specifications, and maintains the equipment.

In addition to the Mays meters maintained in each district, the central office has two trailers along with all testing equipment. One trailer is maintained in a calibrated condition for use by any district in the event of a major equipment problem. The other trailer is used as a calibrated standard trailer and for research purposes.

The total equipment package used in the testing program is as follows:

1. The Mays ride meter, which determines road roughness by measuring vertical movement between the axle and chassis of the test trailer;
2. The roughness trailer, which was designed for use in a roughness-testing program, weighs 800 lb/wheel, and measures 120 in. from axle to hitch;
3. The tow vehicle, which can be any vehicle capable of towing the roughness trailer;
4. The distance-measuring instrument, which electronically measures the distance tested to the nearest 0.001 mile;
5. The speed meter, an electronic unit built by the Georgia DOT to monitor testing speed to within ±0.2 mph; and
6. The digital roughness meter, an electronic display unit designed and built by the Georgia DOT that eliminates the Mays meter chart paper and provides a roughness and testing length readout with one-button operation (results of as many as 63 tests can be stored before the data have to be recorded).

CALIBRATION

The calibration of response-type roughness meters is a weak point of any roughness-testing program unless a true profile of the roadway can be obtained that then can be related to the roughness meter output. Equipment for obtaining such a profile is expensive and is not readily available to many agencies. The other alternative is to use test sections that have been established on in-service roads. This is the system used by the Georgia DOT.

Questions are always raised about the effect of the short-term and long-term increase in roughness of these calibration sections. One way to reduce the effect of these changes is to establish a number of sections on various roadways so that cross checks can be made. Central test sections (CTSs) have been established for calibration reference purposes. These sections are monitored periodically with the central office trailer and serve as an overall calibration standard.

After initial calibration on the CTS, each district established its own field pavement test sections. These sections are tested every 2 weeks to maintain accuracy. The central office calibration control trailer visits each district periodically to check the field test sections and randomly check current projects tested by the district.

Two field test sections are required for each district. Each section consists of a 1-mile length of roadway tested in each direction. One section has a roughness reading in the smooth range and the other section a reading in the medium-roughness range. These sections were carefully chosen to avoid features such as bridges, busy intersections, heavy traffic, and sharp curves.

Each section is initially tested 10 times to establish control limits. A mean roughness value and a range are calculated along with the control limits. Bimonthly calibration checks are plotted on the control charts to determine long-range trends and the calibration history of each road meter. An example of the use of such a control chart is shown in Figure 1. The use of different trailers for construction control dictated that all readings obtained by all trailers on the same section must be comparable. Conversion charts to correct actual readings to adjusted readings to account for calibration differences were deemed inappropriate for use on construction projects. In addition, frequent calibration makes it impractical to determine new calibration graphs often.

To eliminate calibration graphs or multiplying factors, the Georgia DOT is using a mechanical calibration arm to make adjustments. The mechanical adjustment device shown in Figure 2 basically changes the length of the lever arms between the chassis and the axle cables, thereby increasing or decreasing the input to the Mays transducer. This procedure permits fine tuning of the response system and allows each trailer to record identical responses to the road profile. The lever arm also allows for adjustments when a component of the trailer, such as shocks, must be replaced.

Another method of calibration that has been proposed in recent years is the artificial reference surface (ARS) (1). Correlations were made in 1980 by using the ARS and the roughness meters in use in Georgia. Initially, all trailers were adjusted to read the theoretical value of 16.2 in. when driven over the test surfaces (see Table 1). The trailers were then taken to two test sites and roughness readings were obtained with each trailer. The raw data were then corrected by using the appropriate correction equation as prescribed in the ARS procedure. The results of these tests and the corrections given in Table 2 indicate a wide range in test results in both the uncorrected and the corrected data, especially on the smooth section. The process was repeated for two different test sections with the same results.

The next step consisted of using the calibration adjustment arm on each trailer to obtain readings on a smooth road that were as close as possible to a target value. Tests were then made on a road with higher roughness to ensure that the trailers responded to the road profiles in the same way. The results of these tests, given in Table 3, indicate satisfactory agreement between trailers on the smooth and rougher test sections. Based on these test results it was apparent that the ARS method was not sensitive enough for test results obtained on smooth roads and therefore could not be used for calibration purposes by the Georgia DOT.

OPERATING PROCEDURES

The Georgia DOT operating procedures are detailed in a manual that is provided to each operator (2). All tests are made at 50 mph on construction projects with a maximum allowable speed variation of ±1 mph. Tested sections are normally 1 mile long. A minimum of two tests is required for acceptance testing, and the results must be within 10 percent or a third test is required. If none of the test results is within the 10 percent variation limit, the meter is taken back to the calibration section to determine the cause of the problems. Tests are not run when the air temperature is below 32°F; otherwise, no temperature corrections are made to the test results. Roughness caused by bridges and railway crossings is not included in the roadway test results. A Mays meter graph is generally provided only on preconstruction roughness tests, for failing sections during acceptance testing, or at the request of the engineer.

Figure 1. Example control chart for average roughness readings on smooth test section.

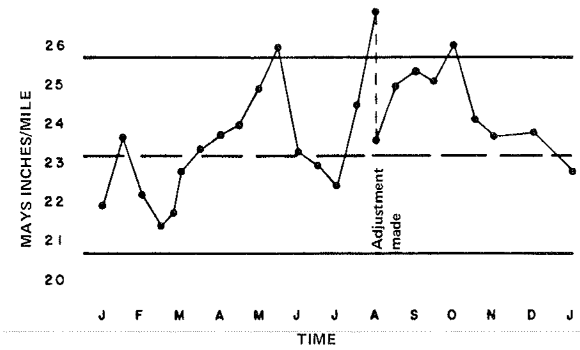
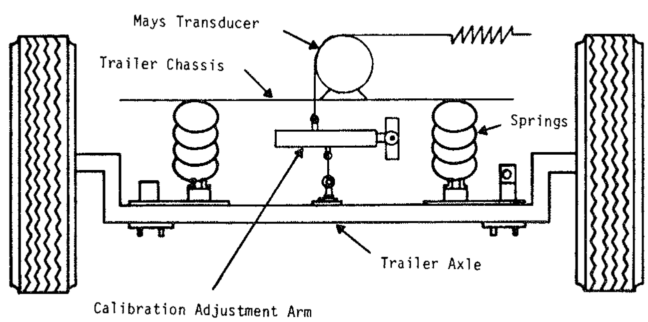


Figure 2. Calibration adjustment arm hookup.



SURFACE TOLERANCE SPECIFICATIONS

For several years Georgia DOT specifications have contained surface tolerance acceptance criteria based on use of the road meter. These roughness specifications have evolved over a period of years. The major steps are

1. Use of the rolling straightedge,
2. PCA meter run for information only,
3. Acceptance based on the PCA meter and straightedge testing of failed sections for penalty assessments, and
4. Use of response-type road meters for acceptance and rejection of projects.

The initial values in the specifications were determined from the information-only results obtained with the PCA meter. Realistic values were set that could be obtained with good construction practices. The Georgia DOT and contractors were familiar with the road meter and the kind of test results that were being obtained at the time the PCA meter was added to the specifications. Throughout the years the roughness limits were lowered as construction equipment and procedures were improved.

In 1980 correlations were obtained between the Mays meter and the PCA meter for the purpose of establishing Mays meter specifications at the same level as those established with the PCA meter.

The system currently in use in Georgia has different specification requirements for concrete pavements, asphalt pavements, and bridge decks. The surface tolerance requirements for asphalt concrete pavement are given in Table 4. The requirements for portland cement concrete pavement are as follows:

Table 1. Results of ARS tests.

		Mays Meter Reading[a] (in./mile)				
Trailer	Date	Both Wheels	Left Wheel Only	Right Wheel Only	Average	Calibration Equation
453	5/30/80	16.2	8.6	7.4	8.0	Y = 0.987X + 2.2
464	6/10/80	16.3	7.6	7.9	7.8	Y = 0.953X + 7.1
465	6/9/80	16.3	8.2	8.1	8.2	Y = 1.000X − 1.1
471	6/6/80	16.2	8.2	8.2	8.2	Y = 1.012X − 2.1
472	6/4/80	16.2	8.8	7.9	8.4	Y = 1.038X − 6.6
473	6/5/80	16.0	8.5	8.5	8.5	Y = 1.080X − 11.6
474	6/5/80	16.5	9.0	8.1	8.6	Y = 1.025X − 7.7
475	6/6/80	16.2	8.6	8.0	8.3	Y = 1.025X − 4.4
476	6/5/80	15.9	7.4	8.2	7.8	Y = 1.000X + 3.2
8967	5/30/80	16.9	9.2	9.1	9.2	Y = 1.052X − 16.9

Note: In tests 453 through 476 the Mays meter was mounted in a trailer; in test 8967 a Torino wagon was used. All trailers were set as close as possible to the theoretical ARS of 16.2 by using the lever arm adjustment. The Torino wagon could not be adjusted.

[a]Reading on ARS x 10.7.

Table 2. Measured roughness versus corrected roughness obtained by using the ARS method.

	Roughness (in./mile)							
	Ga.-7 (Milepost 0-1)				Ga.-362 (Milepost 4.5-5.5)			
	Northbound Lane		Southbound Lane		Eastbound Lane		Westbound Lane	
Trailer	Raw	ARS	Raw	ARS	Raw	ARS	Raw	ARS
453	27.1	28.9	33.0	34.8	56.0	57.5	64.0	65.4
464	20.3	26.4	19.5	25.7	56.3	60.8	54.1	58.7
465	19.8	18.7	22.4	21.3	56.8	55.7	60.0	58.9
471	21.2	19.4	24.3	22.5	56.9	55.5	55.4	54.0
472	22.4	16.7	27.2	21.6	57.5	53.1	56.5	52.0
473	24.1	14.4	27.2	17.8	56.3	49.2	57.4	50.4
474	29.2	22.2	34.8	28.0	61.1	54.9	59.8	53.6
475	37.3	33.8	39.2	35.8	62.1	59.3	62.7	59.9
476	24.2	27.4	28.0	31.2	60.3	63.5	58.4	61.6
Avg	25.1	23.1	28.4	26.5	58.1	56.6	58.7	57.2
Range	17.5	19.4	19.7	18.0	6.1	14.3	9.9	15.0
8967[a]	28.1	12.7	31.7	16.4	69.5	56.2	62.8	49.2

Note: Runs were made immediately after trailers were set on ARS.

[a]Torino wagon.

Location	Measuring Instrument	Roughness (in./mile)
Main line	Mays meter	65
Ramps	Profilograph	14

For ground concrete pavement the specifications require a value of 50 in./mile with the Mays meter and, if the pavement does not pass, a maximum of 7 in./mile with the profilograph. Finally, the specifications for bridge decks are as follows:

Direction	Measuring Instrument	Roughness
Longitudinal	Profilograph	15 in./mile
Transverse	Straightedge	0.2 in./10 ft

The requirements also vary within each pavement type.

These values are presented for information purposes only and would not necessarily be valid for Mays meters installed in vehicles or trailers that have different weights and wheelbases. Asphaltic concrete pavements have different requirements for new construction, open-graded friction courses, and non-Interstate resurfacing. The requirements also contain a target value that is the specification value and roughness levels at which correction of the surface will be required. For portland cement concrete pavements there are different requirements for new construction and ground concrete surfaces. The profilograph is used to determine smoothness on bridge decks and on ground concrete pavement surfaces that fail to meet the Mays meter requirements. The profilograph is used as a secondary acceptance tool on ground concrete surfaces because the grinding equipment can only remove small surface variations over short distances. Grinding of concrete pavement is not done to remove roughness caused by swells, dips, or severe settlement of the pavement, all of which affect road-meter results.

Table 3. Retest of trailers after setting on GA-7.

		Avg Reading[a] (in./mile)			
		Ga.-7		Ga.-3	
Trailer	Date	Northbound	Southbound	Northbound	Southbound
453	6/24/80	22.8	22.4	66.0	59.7
464	6/25/80	22.0	20.7	67.7	64.4
465	6/20/80	23.7	21.0	66.1	61.5
471	6/23/80	20.5	21.7	64.9	59.5
472	6/26/80	23.6	22.7	58.2	53.4
473	6/23/80	22.7	21.0	63.4	59.5
474	6/23/80	24.0	19.8	64.6	60.1
475	6/26/80	22.9	19.6	66.3	60.4
476	6/26/80	21.9	22.6	56.8	55.2
Avg		22.7	21.3	63.8	59.3

Note: All trailers set on Ga.-7 (smooth) and check made to see whether all read the same on Ga.-3 (rougher). Setting on adjusted average of all trailers: 23 northbound and 21 southbound on Ga.-7.

[a]Average of five runs after adjustment.

Table 4. Surface tolerance specifications for asphalt concrete pavements.

	Roughness (in./mile)			
	Open-Graded Friction Courses		Dense-Graded Mixes	
Type of Project	Target Value	Correction Value	Target Value	Correction Value
New construction and Interstate resurfacing	25	30	35	35
Other	25	35	35	45

Note: Applicable to main line and ramps more than 0.5 mile long.

USE OF ROUGHNESS DATA

Since 1972 the Georgia DOT has used a response-type road meter in acceptance of road construction projects for smoothness. The emphasis by the Georgia

DOT on obtaining smooth-riding roads has had a profound effect on the ridability of roads in Georgia. The trends since 1972 are clearly indicated in Figure 3, which shows the statewide roughness averages for the period in which the PCA meter was being used. The preconstruction roughness level has decreased substantially since 1972, and the smoothness levels of new construction and overlays have also improved steadily over the years.

The smoothness requirements forced contractors and field personnel to pay attention to smoothness in paving operations. Better scheduling of trucks, for instance, leads to fewer starts and stops and fewer joints, which cause ride discomfort. The emphasis on obtaining smooth-riding roads is further aided by giving the project engineer roughness results during construction on the leveling, intermediate, and final surface layers. These early results allow for correction during the construction process and result in fewer surprises when the final surface is tested for ridability acceptance.

The recently adopted Mays meter system gives a graphical representation of the roughness input to the meter and can be used by the engineer to determine where leveling is required. In addition, when corrections are to be made to the final surface course, it allows the engineer to pinpoint the locations that need corrective work. An in-depth analysis can be conducted by the engineer or the contractor from the Mays meter graph by plotting the roughness level for each 0.05 mile or any other convenient length versus distance as shown in Figure 4. The graph shows that the roughest section is located between mileposts 6.6 and 6.9. The roughness represented in Figure 4 could be caused by poor construction joints or other problems. This should be verified in the field. Corrective actions could include resurfacing and other methods.

The smoothness levels obtained on construction projects are compiled on a quarterly basis for all completed projects. This report contains rankings for overall smoothness by highway districts and by contractors. The report is distributed throughout the Georgia DOT and to each contractor listed in the report. This fosters a competitive spirit among the highway districts and among individual contractors. The report contains data that compare the roughness values obtained statewide with the specified values, and the roughness obtained for each of the various asphaltic concrete surface mixes is compared.

CONCLUSIONS AND RECOMMENDATIONS

Based on the Georgia DOT experience, the following conclusions have been drawn.

1. The response-type road meter is a rapid, inexpensive instrument that can be used to monitor the ridability of road construction projects.
2. Calibration is a problem and frequent calibration checks are necessary when the roughness meter is used for acceptance or rejection of construction projects.
3. Shock absorbers are the most common reason for roughness meters being out of calibration.
4. Specifications for surface tolerance must be realistic, and the limiting values should be established based on results obtained on projects that have acceptable ride quality.
5. The inclusion of the PCA meter and the Mays meter in the specifications has improved the overall ride quality of Georgia roads.

It is recommended that

1. Research continue on improving calibration

Figure 3. Historical trends of roughness levels in Georgia.

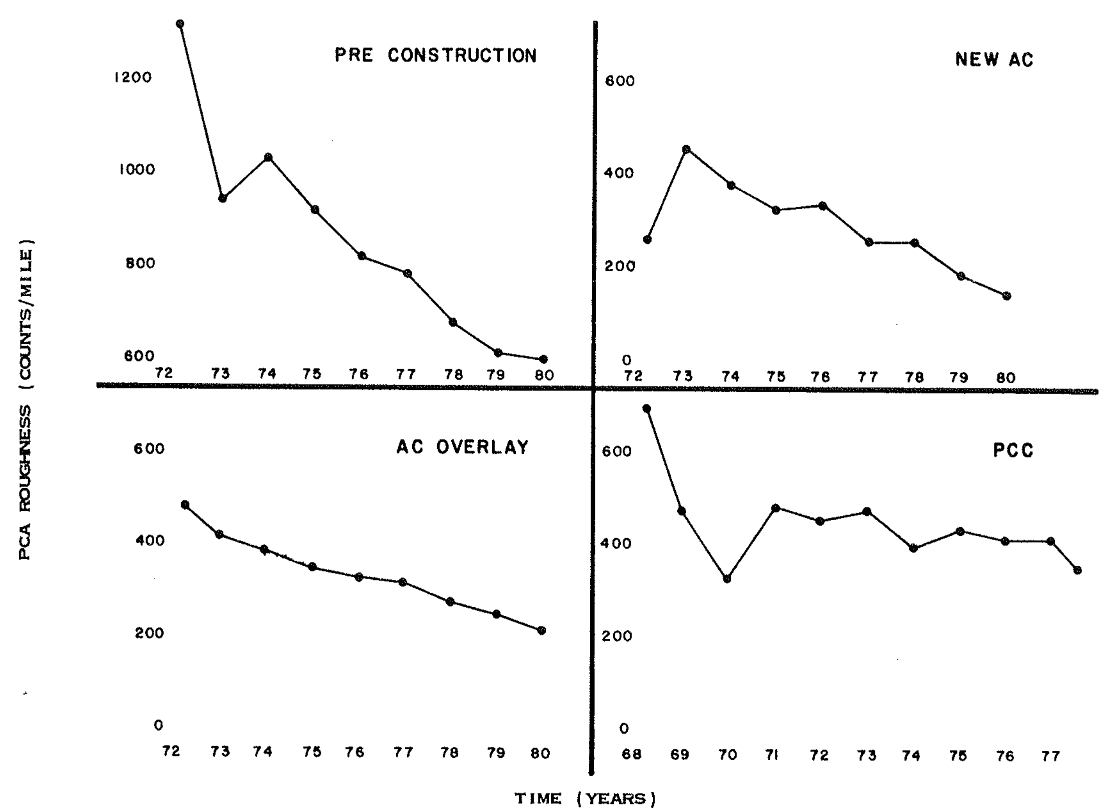
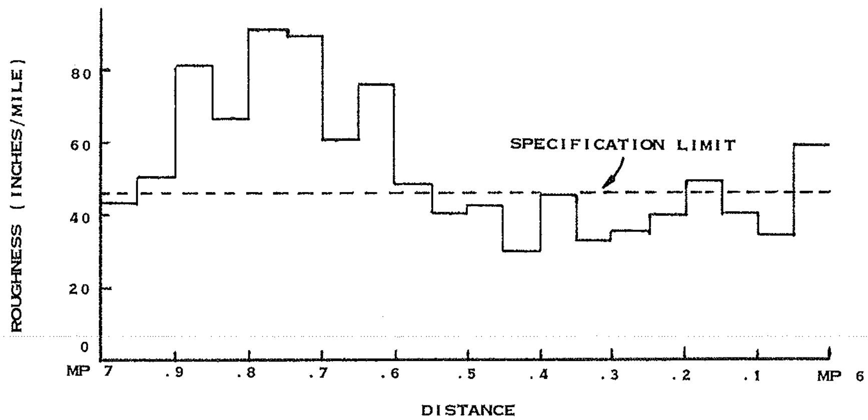


Figure 4. Distribution of roughness plotted from Mays Meter graph.



devices, especially on developing low-cost devices for measuring pavement profiles;

2. High-quality shock absorbers designed for roughness testing be made available; and

3. Any agency that wants to adopt ridability values established by other agencies do so only after correlations have been obtained between the roughness meters of the two agencies.

REFERENCES

1. T.D. Gillespie, M.W. Sayers, and L. Segal. Calibration of Response-Type Road Roughness Measuring Systems. NCHRP, Rept. 228, Dec. 1980.
2. Georgia Mays Meter Control Procedures and Operators Guide. Georgia Department of Transportation, Atlanta, Oct. 1980.

Penn State Automatic System for Collecting and Processing Road Meter Records

M.J. FLEMING, J.C. WAMBOLD, AND G.F. HAYHOE

A microcomputer-based data acquisition and processing system developed as a replacement for the Mays ride meter is described. The system retains the same basic operational characteristics as the Mays meter but offers improvements in resolution, cost-effectiveness, and ease of use and requires a minimum of operator training. System operation is interactive, and the operator is prompted by an alphanumeric display and backlighting of the data input keyboard. Highway event data and road roughness measurements are stored on magnetic digital cassette tape for automatic transfer to a road inventory or pavement management system data base.

The vehicle-mounted Mays ride meter (MRM) is widely used by highway departments to make records of road roughness. These records are used to inspect new construction and to determine the maintenance needs of existing roads. A modification of the commercial MRM system was designed at the Pennsylvania Transportation Institute by Bhargava (1). The system replaced the graphical output of the commercial MRM system with printed numerical output from an onboard computer. The system uses the photocell-based transducer used in the commercial MRM to measure roughness input.

The development of a system that uses an incremental digital encoder as the transducer is described in this paper. The system was developed in cooperation with the Pennsylvania Department of Transportation (PennDOT) to perform onboard processing of the encoder output and to store the resulting measurements of road roughness on a digital cassette tape recorder.

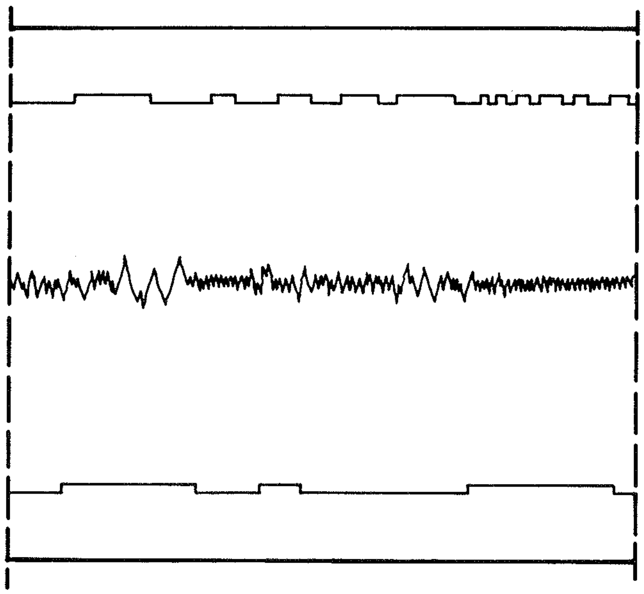
COMMERCIAL MAYS RIDE METER

The commercial MRM operates in a vehicle traveling at highway speeds and is powered by the vehicle's 12-V electrical system. There are two main components of the system: the transmitter, which measures the motion of the rear axle in relation to the vehicle body, and the recorder, which records data from an odometer, an event button, and the transmitter.

The transmitter is attached to the body of the vehicle above the differential. Digital signals sent to the recorder by the transmitter indicate the movement of the rear axle in relation to the vehicle body with a resolution of 0.1 in. (0.25 cm). The transmitter is linked to the rear axle by a rod connected to the differential that senses the vertical position of the rear axle.

The recorder sits on the front seat of the vehicle and is connected to the transmitter by a wire. The recorder traces axle displacement, an odometer mark, and an event mark on a strip chart (see Figure 1). The rate of chart advancement is proportional to the rate of axle displacement so that rough areas of the road generate more output than do smooth areas. Notes can be written on the chart as data are generated (2).

Figure 1. Mays ride meter output.



PENN STATE AUTOMATIC ROAD METER

System Description

The Penn State automatic road meter (PSARM) is a self-contained microcomputer-based system designed to measure road roughness. It also records the type, location, and duration of many different kinds of events or landmarks encountered in testing a specific section of pavement. The device is designed to be installed on the dashboard or the front seat of any car, station wagon, or truck that has a 12-V electrical system. Data are stored on a digital casette tape and can be input directly into a computer-based pavement management system.

The operator provides input through a 32-key pushbutton panel. Each key is backlit with a switched incandescent bulb. A 20-character light-emitting diode (LED) display is used to communicate to the operator the status of the system, test results, and warning conditions if they exist. A 20-character thermal printer provides hard-copy output of the test results. Each of the input and output systems is described in more detail in the following paragraphs. A simplified diagram of the system architecture is shown in Figure 2.

Inputs

Rear Suspension Travel

A rotary incremental digital encoder is mounted on the vehicle to measure the relative motion between the rear axle and the frame of the vehicle. As the relative distance changes, the encoder generates a stream of pulses. Counting the number of pulses generated while the vehicle travels a specified distance gives a direct measure of the total relative axle-body displacement over the count period.

Distance Traversed

Seven symmetrically spaced magnets are mounted on the rim of the left front tire. A proximity sensor generates pulses that are input to a Numetrics 1072 distance meter, which provides a digital readout of the total distance traveled from the beginning of a test on a given section of roadway. Calibrated pulses [each 0.0001 mile (0.00016 km)] are routed from the distance meter to the computer to be used for counting and internal timing.

Pushbutton Inputs

Thirty-two labeled pushbuttons are located on the front panel. These are divided into two 4 x 4 sets. The first set is used to control the test system and includes such functions as power on-off, test begin, test end, test abort, and read in tape. The second set is used for data entry, and the keys are numbered 0 to 9 and lettered A to F. They are primarily used to record events as they occur but also to input numerical data.

Tape Drive

A Memodyne 333C digital tape drive is used to read in a prerecorded tape that contains information about the sites to be tested. When an eight-digit site code is entered on the keyboard at the beginning of each test, the computer searches through the input data and finds the correct header associated with that particular stretch of highway. Testing can then proceed. If an input tape has not been entered, the header search can be bypassed and only the site code is stored.

All tape operations are programmed in software. Rewind, fast forward, recording, and playback modes are included automatically when required. The operator need only insert the tape and engage and disengage the heads when prompted by the LED display.

Outputs

LED Display

A 20-character LED display is used to communicate with the operator. It asks for information and echoes back data entries. The display warns of bad user inputs and notifies the operator when certain system malfunctions occur. It indicates testing progress and shows that information is being properly read from and stored onto tape.

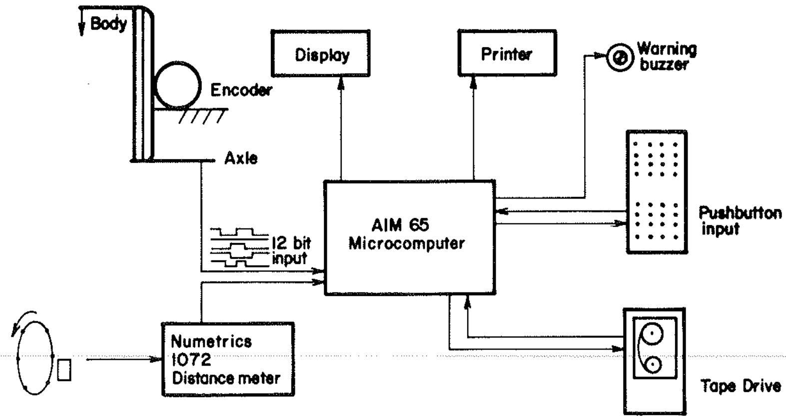
Printer

A 20-character thermal tape printer mounted on the control panel gives a hard-copy summary of the results of each test. Site location and testing conditions are printed, and a chronological list of recorded events (type, location, and duration) also appears. The printer may be deselected; the operator may choose at any time if paper output is to be generated.

Lights Under Pushbuttons

At any given instant, the driver may validly push only a few of the 32 possible buttons, and the sys-

Figure 2. Simplified diagram of PSARM system architecture.



tem will illuminate only those selections that are correct responses to its prompts. When the light is off, the dead-font pushbutton appears as an indistinguishable, unlabeled, textured gray matte. When lit, the color-coded keys display a black, engraved legend.

Warning Buzzer

A small panel-mounted buzzer has been included to attract the driver's attention. A new input may be required, or an important message may be displayed on the readout. Invalid keyboard entries are discouraged by an annoying 2-sec beep. Continuous auditory feedback reminds the driver to close out the recording of an event.

Tape Drive

Accumulated data are dumped to tape at the end of each site testing operation. The software automatically rewinds the tape, advances off the leader and onto oxide, and records the digital information at the same time that it appears on the display. The tape is advanced 3 in. (7.6 cm) between tests.

System Architecture and Hardware

Figure 3 shows the important elements of the system architecture. The PSARM is based on a Rockwell AIM-65 microcomputer. The AIM has been modified and expanded to meet test system requirements. The Microflex expansion system has been used to add extra memory for the temporary storage of tape input and test results. Input-output capabilities have been expanded to accommodate the interfacing of the data acquisition peripherals and the tape drive.

The AIM-65 as delivered consists of two modules (a master module and a keyboard module) interconnected by a plug-in ribbon cable. On the master module are mounted a 20-character LED display and a 20-character thermal tape printer. A comprehensive system monitor is stored in onboard read-only memory (ROM) to drive the various peripherals.

The standard keyboard module on the AIM-65 is a simple switch array that is scanned by an R6532 RIOT chip. Machine-code subroutines in the monitor are used to read inputs. Because the ASCII keyboard is impractical to use in the test system, it has been replaced by a custom switch array especially designed for use by the PSARM. The 32 test system pushbuttons that serve as inputs replace elements in the array supplied by Rockwell (see Figure 4). The new switch array is connected to the master module by the same 16-pin dual in-line package plug used by the standard keyboard module. The input routines in the monitor are not altered by this change.

The display and printer are attached to the

Figure 3. PSARM system hardware.

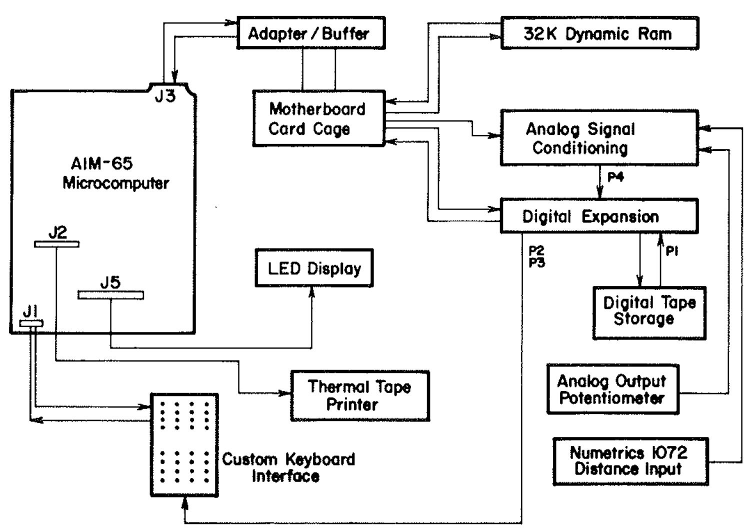
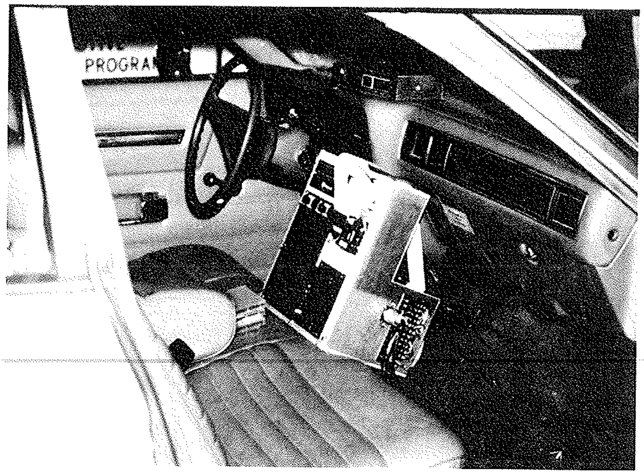


Figure 4. PSARM control panel.



master module at the factory. They have been relocated in the PSARM system so that they can be mounted on the front panel of the test system control module. Short, flat cable jumpers are used to transmit information to the new mounting positions.

Program Storage

The operating system is written in 6502-based assembly language and is stored in ROM on the AIM-65 board. This ROM chip replaces the optional assembler and uses a vectored starting location at DOOH. Program size is slightly less than 4,000 bytes.

System Operation

When the power is turned on, the system is initialized and comes up in the AIM-65 monitor. Depression of the RESET key starts execution at the beginning of the control program. An infinite loop ensures that the only possible exit from the testing software is a complete power-down of the system.

Before testing can begin four activities must take place:

1. The user must enter identification information, such as operator number, vehicle number, and date.
2. The distance-recording system must be calibrated. The software consists primarily of a set of instructions to be used with the Numetrics 1072 distance meter.
3. The axial displacement system must be calibrated. This involves testing over a known surface. The results are then compared with known values to arrive at a calibration constant that will yield outputs in engineering units.
4. The input tape, which contains information about the sites to be tested that day, is read in.

The system is now fully prepared for operation. It waits in a stand-by mode for testing to begin.

When the TEST BEGIN key is depressed, the computer asks for information about the next site to be tested. Data for the site code, section numbers, starting milepost, speed, direction of travel, and testing lane are entered from the keyboard. The depression of the START key signals the beginning of the test. The only requirement of the driver is that he maintain a testing speed close to the previously entered value.

The user may choose at any time to log in an event or landmark as it occurs during testing. The type of event is selected by depressing one of the precoded event keys on the hexadecimal keypad. The exact location of the beginning of the event is recorded by pushing the START-STOP key. A second depression stores the ending point. All distances are referenced from the beginning of the test.

When different types of roads are to be tested, it is possible that unique sets of events will be encountered. Depending on the type of road, the driver may choose any 1 of 6 sets of 16 events for any given test. The correspondence of events to input keys is completely arbitrary within a given set and is precoded by a clear plastic overlay similar to the ones used for programming scientific calculators. Before a test the driver enters the overlay code (labeled A-F) for the set of events to be used.

For all of the overlay codes, event key F is used to identify a discontinuity in the numbering of mile markers. The first mile marker of the renumbered sequence is entered from the keypad, and its exact distance from the beginning of the test is stored.

The end of the test is signaled by the TEST END key. The computer advances the tape and records the roughness data and any information about events that have been logged. After the last test of the day, the digital data tape is removed from the recorder and the system is turned off.

Data Acquisition System

Road roughness information is continuously collected and reduced as long as the system is turned on. The control system determines when (and for how long) the data will be permanently saved to be recorded later on tape. All of the peripherals associated with data acquisition are slaved to the central processing unit (CPU). A single interrupt is generated to the CPU when any of these devices need to be serviced. When an interrupt occurs, the program control immediately transfers from the control system to a handler routine that determines the nature of the disturbance. The highest-priority interrupt in the data acquisition process is the 0.0001-mile pulse from the distance meter. When this interrupt has been acknowledged by the handler routine, control passes to a subroutine that reads the encoder pulse counter, adds the count value to a temporary accumulator, resets the counter, increments a distance pulse accumulator, and checks whether data for 0.05 mile (0.08 km) have been recorded. On overflow of the 0.05-mile accumulator, the value in the encoder pulse accumulator is transferred to data memory, the accumulator is set to zero, and the distance traveled from the start of the test count is updated. Otherwise, control is passed back to the main program to await the next distance pulse interrupt or a key closure signifying that an event has occurred.

When an event has been acknowledged, the event code is stored. The value of the total-distance-traveled counter is stored in temporary memory on the first and second closures of the START-STOP switches. All relevant event information is then formatted and stored in data memory with an event-identifying header.

Preliminary Test Results

The system has been tested by PennDOT on selected sites that have a wide range of road roughness. The equipment was mounted in a PennDOT road-meter vehicle, and the encoder was driven directly from the shaft of a rotary Mays transducer already installed in the vehicle so that direct comparison with the Mays meter chart output was possible.

Linear regression of the PSARM results versus the

Mays meter results gave a correlation coefficient of 0.99. Both systems therefore have almost identical characteristics over a wide range of operating conditions except that the PSARM shows better resolution and automatically reduces and stores the data. In the test configuration, the resolution of the axle displacement measurement was approximately 100 times better with the PSARM system than with the Mays meter.

CONCLUSIONS

A microprocessor-based data acquisition system is the basis of the PSARM, which has been developed as a replacement for the Mays ride meter currently used by many state highway departments. The system offers substantial improvements in resolution, cost-effectiveness, and ease of use and requires a minimum of operator training. It provides all of the functions currently found on the Mays meter and adds several important features:

1. Strip-chart output, which is expensive to reduce, has been eliminated in favor of reduced data storage on digital cassette tape. Information can now be fed directly into a road inventory data base. Summary data are provided in hard copy as the test proceeds.
2. The event identification and recording procedure has been streamlined, and more information is obtained. The type, location, and duration of any event are stored directly onto tape with a minimum of user input.

ACKNOWLEDGMENT

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REFERENCES

1. R.K. Bhargava. Automating the Data Processing of the Mays Ride Meter. Department of Mechanical Engineering, Pennsylvania State Univ., University Park, 1979.
2. The Incomparable Mays Ride Meter for Pavement Surveillance. Rainhart Co., Austin, Tex., 1972.

Mechanistic Model for Predicting Seasonal Variations in Skid Resistance

KAZUO SAITO AND J. J. HENRY

Some of the findings of a 3-year research program to develop a basic mechanistic model for predicting seasonal and short-term variations in skid resistance as a function of environmental and traffic conditions are described. The model treats seasonal and short-term variations separately. Data from 21 test surfaces in State College, Pennsylvania, and 10 surfaces in Tennessee and North Carolina were analyzed. For the seasonal trend, an exponential curve was fitted to the skid number data for the asphalt pavements whereas a linear relationship best fit the data for portland cement concrete surfaces. The coefficients of the resulting seasonal variation curves were related to pavement and traffic parameters to provide predictors for long-term effects. Significant predictors were found to be British pendulum number (BPN) and average daily traffic. Other predictors for pavement polishing are suggested in place of BPN to predict the rate of decrease in skid resistance over an annual cycle. After the data for seasonal variations were adjusted, the remaining short-term variations were regressed against rainfall, temperature, and macrotexture parameters. The short-term variations can be predicted by dry spell factor and pavement temperature, but the introduction of the measured percentage normalized gradient was found to improve the regressions. Although good agreement was observed for the test data from the two locations, it is suggested that similar investigations be conducted in other geographic areas.

It is generally recognized that the skid resistance of pavement surfaces changes with time. Two decades ago, Giles and Sabey (1) reported that investigations on some British pavements revealed the existence of significant differences between summer and winter skid resistance. They also presented data that showed a strong relationship between seasonal variation in skid resistance and personal-injury accidents.

Skid-resistance measurements made on public highways in Pennsylvania and other states in accordance with ASTM E274 (2) exhibit seasonal and short-term variations (3-5). Seasonal cycles have been observed in the northern states, where skid resistance tends to be higher in winter through spring than in summer through fall. Superimposed on these annual cycles are short-term variations that appear to result from rainfall and other local weather conditions. These variations make it difficult to establish a rational maintenance program in which skid resistance is one of the important factors.

During the past two decades, several transportation departments and other agencies in the United States have conducted extensive skid-resistance surveys, but until the past few years little attention was paid to seasonal variations in these measurements. Until recently, the most comprehensively documented studies involving both seasonal and short-term skid-resistance variations were the ones undertaken by the Pennsylvania Department of Transportation (4,5). The skid-resistance measurements made in the first of these studies showed that, when the pavement surfaces had stabilized after being exposed to weather and traffic for 1 or 2 years, they exhibited cyclic skid-resistance variations. Several other states have reported to FHWA their observations on seasonal skid-resistance variations. Extreme seasonal variations as high as 30 skid numbers have been observed as well as more typical variations in the range of 5 to 15. These observations were summarized in 1977 by Rice (6). Analyzing these large changes, which occur rather systematically, Hegmon (7) concluded that there are real changes in skid resistance that are related to changing conditions.

Observed seasonal and short-term variations in skid resistance make it difficult for a state transportation agency to determine the minimum skid-resistance value for a given road surface. It is obviously impossible for the states to conduct their entire inventory program in the short period in which pavement skid resistance is expected to be at its lowest value. Some states conduct skid tests during most of the year, except for periods of freezing weather. Where the testing season is short, it may require several years to conduct a complete inventory of a state highway system. Thus, there is a need to establish analytic procedures that provide corrections to measured skid resistance for seasonal and short-term variations in test conditions.

The observed skid-resistance variations reported by various agencies are helpful in providing qualitative information on the trends and magnitude of seasonal and short-term variations in skid resistance. However, the measurements have not been taken frequently enough to provide the information needed to develop a model that could be used to predict the lowest skid number expected during the year on a given pavement.

FHWA recognized the need for analytic means of interpreting skid-resistance data subjected to seasonal and short-term variations. In 1978 FHWA initiated a 3-year research program with the Pennsylvania Transportation Institute of Pennsylvania State University to collect frequent skid-resistance measurements of pavements in various geographic areas of the United States and to develop predictor models for describing seasonal variation in skid resistance.

In this paper, the findings of a portion of this research program are described--i.e., the development of a basic mechanistic model for predicting seasonal and short-term variations in skid resistance as a function of environmental and traffic conditions. The model is based on 21 pavements in Pennsylvania and 10 pavements in Tennessee and North Carolina. Complete results of the project are reported elsewhere (8).

DATA BASE

The data base consisted of skid-resistance measurements taken at various speeds, pavement-related data, and weather data recorded at weather stations located near the test sites.

Test Sites

Skid testing was performed on 21 test pavements in Pennsylvania between January and December 1980. The 21 sites represented a variety of aggregates and mix designs and included 16 asphalt pavements and 5 portland cement concrete (PCC) pavements, which were subjected to a wide range of average daily traffic (ADT). During the same period, data were collected for 10 sites in Tennessee and North Carolina. The pavement and traffic parameters for each site are given in Table 1. The construction materials and locations of the test sites have been fully described by Henry and Dahir (9).

Skid-Resistance Tests

For the Pennsylvania sites, the daily skid-resistance tests were made in the transient slip mode (10). These tests provided SN_{64} data according to ASTM E274 and also brake slip numbers at 16, 48 km/h (10, 20, and 30 mph), which can be use approximate SN_{16}, SN_{32}, and SN_{48} (10). For the Tennessee and North Carolina sites, locked-wheel skid-resistance measurements were conducted primarily at 64 km/h (40 mph), although some tests were made at 48 and 80 km/h (30 and 50 mph). Air, tire, and pavement temperatures were recorded at the time of each test.

Texture Measurements

Monthly texture measurements made at each site included British pendulum number (BPN) according to ASTM E303 (2) and mean texture depth (MTD) according to the sand-patch method described by the American Concrete Paving Association (11).

Weather-Related Data

The weather data available in the daily data base for Pennsylvania sites were obtained from weather records provided by the Pennsylvania State University weather station at University Park. For North Carolina and Tennessee sites, information was obtained from weather stations at Ashville, North Carolina, and Knoxville, Tennessee.

Pavement Polishing Data

During July 1980 the Penn State reciprocating pavement polisher (12) was used in a series of tests carried out on the Pennsylvania sites. The device uses a loaded rubber pad (100 by 150 mm) through which a slurry containing abrasive is introduced. Each pavement was subjected to 2,000 cycles of polishing with a 0.05-mm silica abrasive; measurements were taken initially (BPN_0), after 500 cycles (BPN_{500}), and after 2,000 cycles (BPN_{2000}). The polishing was performed on unpolished portions of the pavement (out of wheel tracks). The results are given in Table 2. In many cases the BPN values were higher after 500 cycles of polishing than initially; this is thought to be a result of the removal of the surface glaze.

DEVELOPMENT OF MECHANISTIC MODEL

In the course of evaluating the data collected in the research program, some cyclic patterns were observed. Measurements showed that the seasonal variations from spring to fall were similar for all of the bituminous pavements: the skid number was low in the fall and was brought to approximately its original level as skid resistance was rejuvenated over the winter season (see Figure 1). Superimposed on this seasonal cycle were short-term variations that resulted in low skid numbers after a dry period and high (rejuvenated) skid numbers after a rainy period (see Figure 2) (3,13,14). These trends indicated that it might be possible to develop an equation or a model to predict the low skid numbers that occur in the fall from a skid-resistance measurement taken at any time during the year.

Based on these observations, a mechanistic model that treats seasonal and short-term variations separately has been developed (15-17). In this model it is hypothesized that seasonal variation is due to a reduction in the microtexture and the macrotexture as a result of polishing and wear of the aggregate. Polishing causes a reduction of microtexture, and wear results in a reduction of macrotexture. The short-term effects are attributed to contaminants that accumulate on the pavement (18) and, in some cases, to chemical reactions such as those that might occur between limestone aggregate and acid rain. The short-term effects, therefore, have been

Table 1. Pavement and traffic parameters.

State	Site	Type of Pavement	Year of Construction	Type of Aggregate: Coarse	Type of Aggregate: Fine	Percentage Normalized Gradient (h/km)	British Pendulum No.[a]	Mean Texture Depth[a] (mm)	ADT (no. of vehicles)
Pennsylvania	1	DG	1970	Limestone	NA	0.83	58.5	0.368	6,630
	2	PCC	1960	Limestone	Natural sand	0.32	53.0	0.394	7,700
	3	PCC	1973	Limestone	Natural sand	0.71	70.0	0.330	3,640
	4	DG	1972	Limestone	NA	0.84	62.5	0.330	3,640
	8	DG	1972	Limestone	Silica sand	0.61	55.0	0.864	1,820
	9	DG	1972	Limestone	Silica sand	0.69	69.5	0.645	1,710
	10	PCC	1973	Limestone	Silica sand	0.77	72.0	0.292	1,710
	11	DG	1963	Limestone	NA	0.79	56.0	0.432	4,490
	12	DG	1970	Limestone	NA	0.63	60.0	0.648	4,490
	13	OG	1969	Limestone	NA	0.53	90.5	0.978	7,920
	14	PCC	1967	Limestone	NA	0.83	62.0	0.368	8,770
	15	OG	1969	Limestone	NA	0.53	86.5	1.194	7,920
	16	DG	1966	Limestone	Limestone	0.88	50.0	0.394	6,500
	17	DG	1961	Limestone	Limestone	0.67	53.5	0.745	800
	18	PCC	1973	Limestone	NA	0.66	77.0	0.470	1,200
	19	DG	1968	Limestone	Silica sand	0.81	54.0	0.508	7,000
	20	DG	1968	Limestone	Silica sand	0.82	65.0	0.508	7,000
	21	OG	1969	Limestone	Silica sand	0.68	64.0	1.029	2,500
	22	OG	1969	Gravel	Silica sand	0.58	84.5	1.384	2,500
	24	DG	1963	Limestone	NA	0.83	54.0	0.432	4,490
	25	DG	1963	Gravel	NA	0.68	81.0	0.521	7,920
Tennessee and North Carolina	1	DG	1976	Gravel	Natural sand	0.49	73.3	0.945	2,377
	2	DG	1968	Gravel	Natural sand	0.41	86.3	1.600	2,107
	3	PCC	1967	Limestone	Natural sand	0.81	56.8	0.785	11,347
	4	DG	1976	Gravel	Natural sand	0.44	68.2	1.186	4,610
	5	DG	1962	Limestone	Natural sand	0.43	54.0	1.389	1,773
	6	BST	1967	Limestone	Natural sand	0.38	57.2	1.344	640
	7	DG	1970	Granite	Granite	0.50	71.0	0.856	3,973
	8	PCC	1971	Gravestone	Gravestone	0.60	58.2	1.524	6,475
	9	BST	1972	Gravestone	NA	0.41	75.2	1.636	2,310
	11	DG	NA	Slag	NA	0.70	12.0	1.034	4,354

Note: DG = dense graded, OG = open graded, and BST = bituminous surface treatment.
[a]Average value of tests made in April and May.

Table 2. Results of polishing tests with Penn State reciprocating pavement polisher for Pennsylvania sites: 1980.

Site	BPN: Initial (BPN_0)	BPN: After 500 Cycles (BPN_{500})	BPN: After 2000 Cycles (BPN_{2000})	$\frac{BPN_{500} - BPN_{2000}}{BPN_{500}}$ (%)
1	59	60	59	1.67
2	68	75	64	16.00
3	74	79	70	11.39
4	58	68	64	5.88
7	68	70	71	-1.43
8	56	51	50	1.96
9	71	66	69	-4.55
10	70	72	75	-4.17
11	67	68	66	2.94
12	87	82	73	10.98
13	89	85	87	-2.35
14	73	68	66	-4.17
15	87	85	81	4.71
16	70	62	56	9.68
17[a]	–	–	–	–
18	74	73	67	-1.61
19	65	62	63	8.22
20	65	62	63	-1.61
21	67	74	68	8.11
22	81	76	78	-2.63
24	50	59	56	5.08
25	79	77	71	7.79

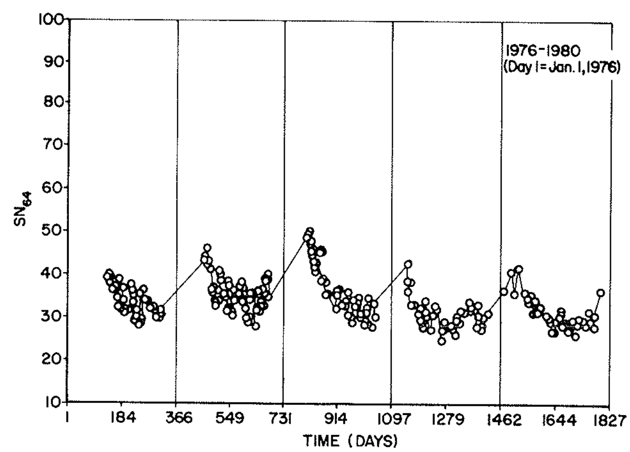
[a]Site has been resurfaced.

modeled as causing short-term modifications to the microtexture.

The model uses the Penn State model (19), in which SN_0 is related to microtexture and PNG is related to macrotexture:

$$SN_V = SN_0 \exp[-(PNG/100)V] \quad (1)$$

Figure 1. Five-year history of skid-resistance variations with time for Pennsylvania site 16 (dense-graded asphalt).



where

SN_V = skid number at velocity V (km/h);
SN_0 = skid number/speed intercept, which correlates well with microtexture; and
PNG = percentage normalized gradient (h/km).

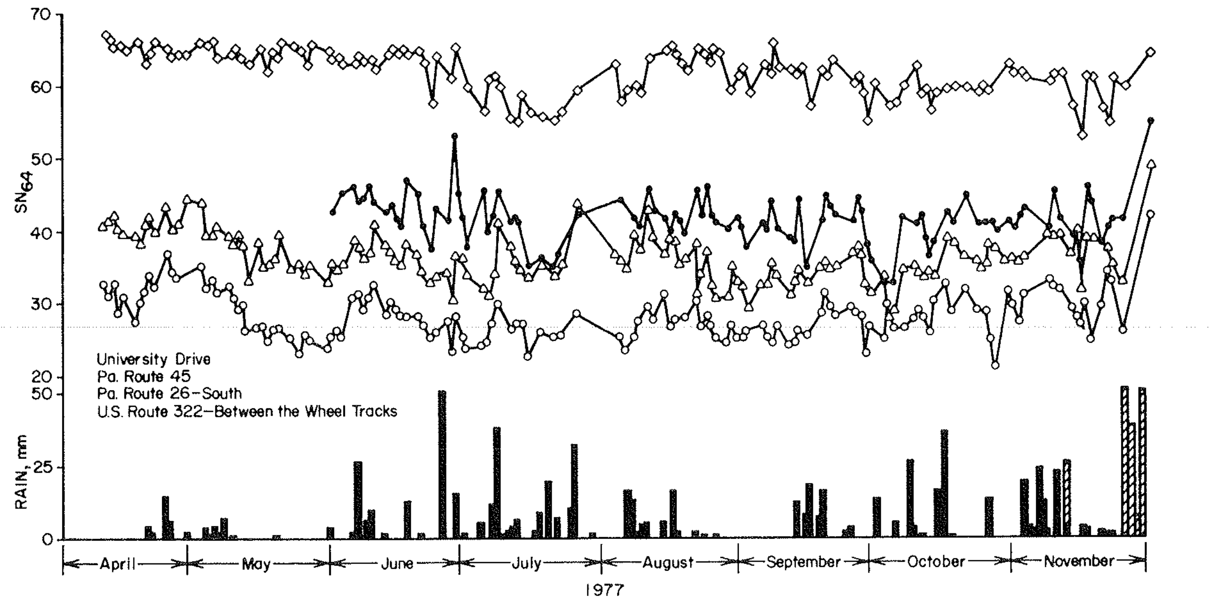
PNG, defined as $-(100/SN) \cdot [d(SN)/dV]$, correlates well with macrotexture.

For skid resistance at 64 km/h (40 mph),

$$SN_{64} = SN_0 \exp(-0.64PNG) \quad (2)$$

The term SN_0 (microtexture) has both seasonal and short-term components (SN_{0L} and SN_{0R}), where Sn_{0R} is the residuals after curve-fitting a sea-

Figure 2. Skid number (SN_{64}) and rainfall data for Pennsylvania sites: 1977 test season.



sonal trend SN_{0L}. Thus, the value of SN_0 at any time can be expressed as

$$SN_0 = SN_{0L} + SN_{0R} \tag{3}$$

The values of SN_0 deduced from data collected throughout the year typically exhibit seasonal variations, as shown in Figures 3 and 4. Figure 3 shows the results for dense-graded asphalt cement surfaces. The seasonal trend for this case can be considered to be exponential in nature, whereas the trend in the data for PCC surfaces (Figure 4) is linear.

For asphalt surfaces, the seasonal component is well-described by an exponential relationship at any time t when a measurement is made:

$$SN_{0L} = SN_{0F} + \Delta SN_0 \exp(-t/\tau) \tag{4}$$

For PCC surfaces, however, a linear relationship better fits the observations:

$$SN_{0L} = SN_{0F} + (\Delta SN_0/\tau)(\tau - t) \tag{5}$$

where

SN_{0F} = level of SN_0 after the pavement is fully polished (SN_{0F} is independent of both seasonal and short-term variations);

ΔSN_0 = polish susceptibility of the aggregate (an aggregate property); and

τ = polishing rate of the aggregate, a combination of aggregate property and ADT.

At any time t when a measurement of SN_{64} is made, Equations 2 through 4 combine for asphalt pavement surfaces to yield

$$SN_{64} = [SN_{0R} + SN_{0F} + \Delta SN_0 \exp(-t/\tau)] \exp(-0.64PNG) \tag{6}$$

The level of skid resistance at the end of the season (SN_{64F}) can be written as follows (note that the mean of the residuals SN_{0R} is zero):

$$SN_{64F} = SN_{0F} \exp(-0.64PNG) \tag{7}$$

Figure 3. SN_0 versus time for dense-graded Pennsylvania site 8: 1980.

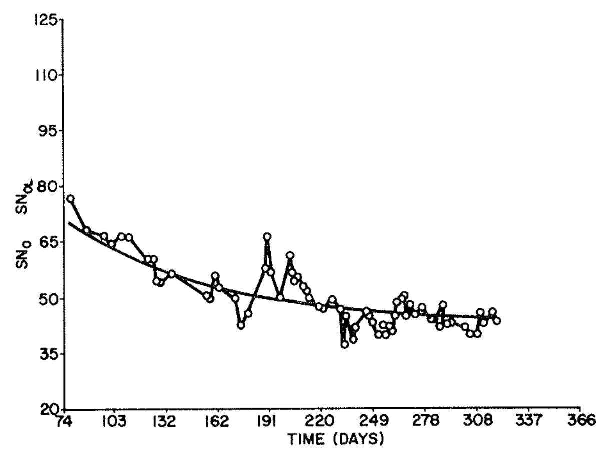
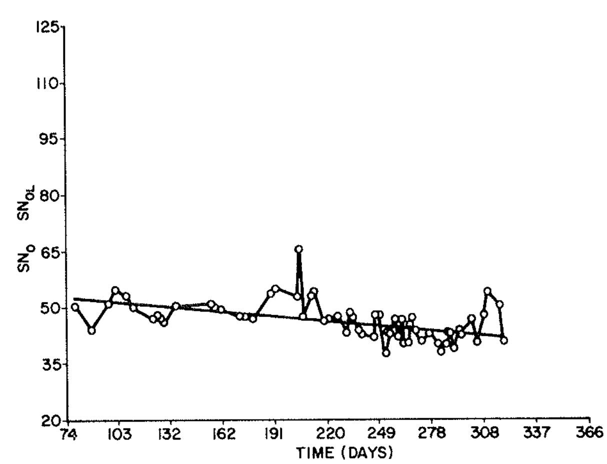


Figure 4. SN_0 versus time for PCC Pennsylvania site 2: 1980.



Substituting Equation 7 into Equation 6 to eliminate SN_{0F} and rearranging produces a relationship that can be used to predict the level of skid resistance at the end of the year (SN_{64F}) from a measurement taken at any other time during the season (SN_{64}):

$$SN_{64F} = SN_{64} - [SN_{0R} + \Delta SN_0 \exp(-t/\tau)] \exp(-0.64PNG) \quad (8)$$

For PCC surfaces,

$$SN_{64F} = SN_{64} - [SN_{0R} + (\Delta SN_0/\tau)(\tau - t)] \exp(-0.64PNG) \quad (9)$$

The short-term component SN_{0R} in Equation 3 can be described by variables related to weather and texture in the form of the following linear model:

$$SN_{0R} = a_0 + a_1 x_1 + a_2 x_2 + \ldots + a_n x_n \quad (10)$$

where a_i is a coefficient determined by multiple regression and x_i denotes variables related to weather and texture.

FITTING OF SEASONAL RELATIONSHIP

The reduction in the skid resistance of asphalt pavements that occurs over the testing season appears to be exponential. Simple regression techniques cannot be used to fit an exponential relationship to the sequence of all data points in the form of Equation 4. There are three variables for which values are to be found: SN_{0F} (the magnitude of SN_0 at $t \to \infty$), ΔSN_0 (the difference between the intercept at $t = 0$ and SN_{0F}), and τ (time constant for the exponential decrease).

To overcome this problem and to fit the data systematically, for each site data were first averaged for each month, and these average values of SN_0 were assigned at the middle of each month. Next, the seasonal variations of monthly averages of SN_0 were fit according to the shifted model instead of Equation 4 because the highest recorded values of SN_0 at each site were observed at the beginning of the season, in mid-March ($t = 74$ days):

$$SN_{0L} = SN_{0F} + \Delta SN_0 \exp[-(t - 74)/\tau] \quad (11)$$

Figure 5 shows the basic concept of this model, and Figures 6 and 7 show the procedure used to obtain the best fit of the monthly averaged data to Equation 11. The value τ is treated as an independent variable, and the data are regressed for a fixed value of τ. To fit the data, τ is varied and the data are regressed to provide values of SN_{0F} and ΔSN_0 for each value of τ. When there is sufficient traffic and the pavement has enough polish susceptibility to reduce the value of SN_{0F} to its terminal value (Figure 6), the regression that provides the highest correlation is selected (see Figure 8). When the terminal value of SN_{0F} is not reached within the season (Figure 7), a maximum correlation coefficient does not occur for τ less than 290 days. In this case the criterion is to select a data set that corresponds to an improvement of the correlation coefficient (R^2) of less than 0.001 for a 10-day increment of τ (see Figure 9). The results for all asphalt concrete surfaces from the Pennsylvania, Tennessee, and North Carolina sites are summarized in Table 3.

For PCC surfaces, the data exhibited a linearly decreasing trend in SN_0 with time over the testing season. The following linear model was applied to yield the average value of SN_{0F} and the rate of decrease ($\Delta SN_0/\tau$), where τ is fixed at 275 days (mid-December):

$$SN_{0L} = SN_{0F} + (\Delta SN_0/\tau)(\tau - t + 74) \quad (12)$$

These results for Pennsylvania sites are also given in Table 3.

Figure 5. Basic concept of mechanistic model.

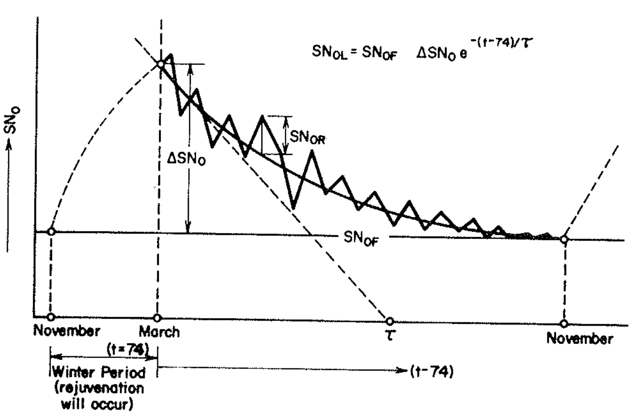


Figure 6. Annual variation of SN_{OL} for site whose terminal value is reached.

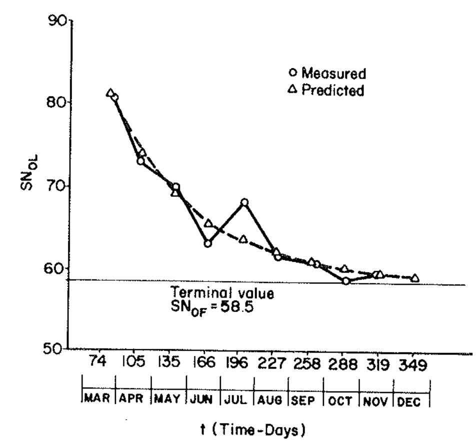


Figure 7. Annual variation of SN_{OL} for site whose terminal value is not reached because of insufficient polishing.

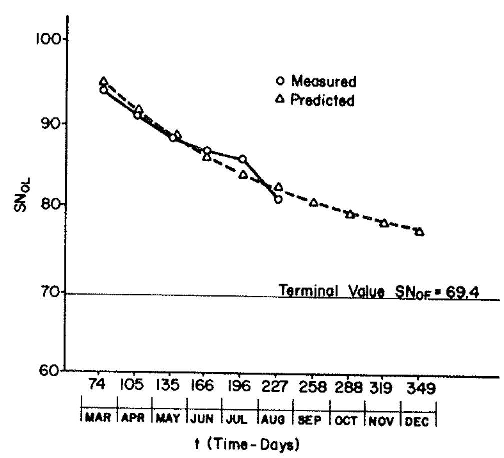


Table 3. Parameters of model for seasonal variations in skid resistance.

State	Type of Surface	Site	τ	ΔSN_0	SN_{0F}	R^2
Pennsylvania	Asphalt	1	190	22.8	44.2	0.765
		4	160	26.5	47.9	0.848
		8	80	28.6	43.1	0.919
		9	40	28.0	64.9	0.672
		11	110	19.4	44.2	0.787
		12	210	32.5	46.9	0.795
		13	160	26.6	78.4	0.926
		15	210	31.0	77.0	0.939
		16	170	14.8	34.3	0.656
		17	130	26.4	40.0	0.750
		19	140	19.9	44.2	0.844
		20	90	23.1	57.6	0.893
		21	150	26.2	40.4	0.767
		22	170	32.5	66.7	0.866
		24	190	20.4	39.6	0.720
		25	210	25.3	69.4	0.963
	PCC	2	275	12.4	40.5	0.544
		3	275	11.5	66.7	0.546
		10	275	8.2	77.8	0.512
		14	275	9.6	60.6	0.597
		18	275	5.4	73.0	0.323
North Carolina and Tennessee	Asphalt	1	390	21.2	60.0	0.695
		2	500	23.5	50.3	0.545
		4	250	28.4	53.6	0.746
		5	190	7.8	38.1	0.411
		6	50	10.2	39.1	0.672
		7	20	20.9	70.4	0.508
		9	100	11.4	62.7	0.791
		11	210	23.9	47.3	0.803
	PCC	3	275	25.9	35.2	0.514
		8	275	3.2	52.2	0.046

PREDICTION OF SEASONAL PARAMETERS

After the values of SN_{0F}, ΔSN_0, and τ were obtained from measured data, methods of predicting them were tried. SN_{0F} is a measure of the microtexture of the pavement after removal of the seasonal and short-term effects. Thus, it seemed likely that a microtexture parameter could be used to predict SN_{0F}. Monthly measurements of BPN were available for each of the test pavements. A regression of SN_{0F} versus BPN (the average value of measurements in April and May) for asphalt pavement surfaces (see Figure 10) yields the following. For the Pennsylvania sites

$$SN_{0F} = -16.32 + 1.068BPN \quad R = 0.989 \tag{13}$$

For the Tennessee and North Carolina sites

$$SN_{0F} = -33.78 + 1.281BPN \quad R = 0.983 \tag{14}$$

A regression for PCC surfaces in Pennsylvania yields

$$SN_0 = -32.83 + 1.445BPN \quad R = 0.938 \tag{15}$$

The ΔSN_0 parameter is a measure of the rejuvenation of skid resistance (Figure 5) that occurs during the winter months as a result of the depolishing effects of winter conditions (5) and is also a measure of the polishing susceptibility of the aggregate by traffic. Therefore, BPN and ADT seemed likely parameters to be used as predictors. A linear regression of ΔSN_0 versus BPN and ADT for asphalt pavement surfaces (see Figure 11) yields the following. For Pennsylvania,

$$\Delta SN_0 = 6.69 + 0.324BPN - 0.000852ADT \quad R = 0.921 \tag{16}$$

For Tennessee and North Carolina,

$$\Delta SN_0 = -15.31 + 0.369BPN + 0.00348ADT \quad R = 0.944 \tag{17}$$

Figure 8. Procedure for determining best fit of data in Figure 6.

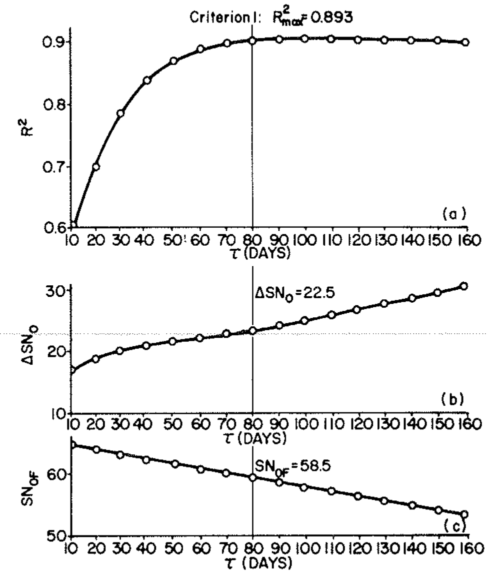
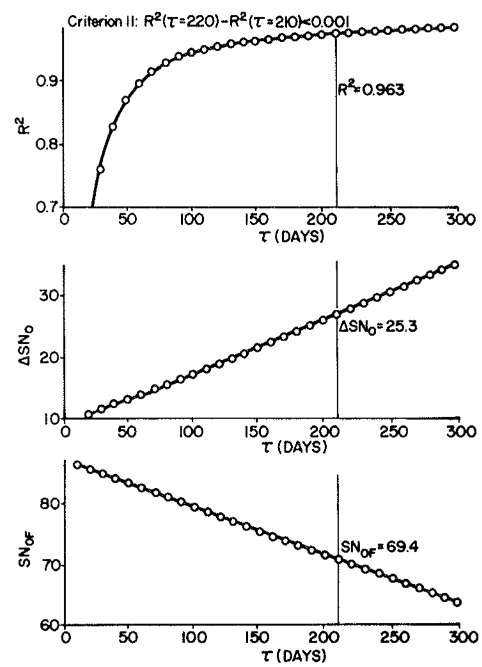


Figure 9. Procedure for determining best fit of data in Figure 7.



A regression for PCC surfaces in Pennsylvania yields

$$\Delta SN_0 = 29.51 - 0.289BPN - 0.000171ADT \quad R = 0.796 \tag{18}$$

The results for Pennsylvania sites indicate that the depolishing of the pavement that occurs as a result of the use of winter deicing chemicals is offset by the mechanical polishing that occurs with moderate traffic volumes. At the Tennessee and North Carolina sites, the effect of the depolishing of the pavement in winter is less apparent because the aggregate is polished further by the mechanical polishing that occurs with larger traffic volumes.

The mechanical aspects of pavement rejuvenation become important when the winter use of studded tires is considered. Data are available for five of the asphalt pavements in Pennsylvania for a period of three consecutive winters. In the winter of the second year (1978-1979), the use of studded tires was prohibited. The data given in Table 4 show that ΔSN_0 is consistently greater for the two winters during which studded tires were used. Specifically, ΔSN_0 is greatest for the first winter, during which studded tires were used by a large number of motorists, and for the third winter, during which studded tires were used by a relatively small number of motorists because it was uncertain until late November whether the use of studs would be permitted. These results appear to support the theory that a significant factor in winter rejuvenation of the surface texture is the mechanical interaction between tire and pavement.

Figure 10. SN_{OF} versus BPN for asphalt pavement surfaces.

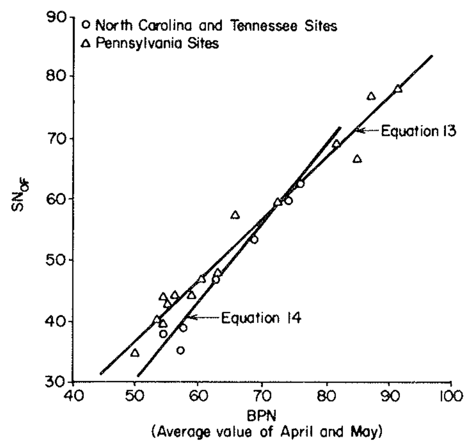


Figure 11. Prediction of ΔSN_0 from BPN and ADT for asphalt pavement surfaces.

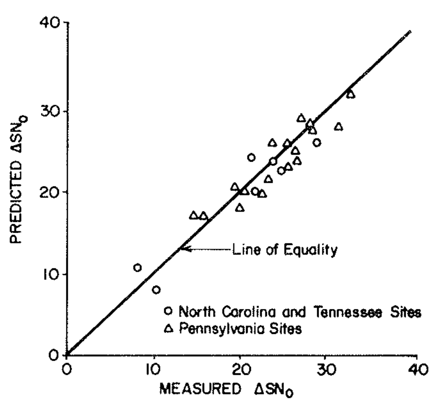


Table 4. Polish susceptibility of aggregate (ΔSN_0) for six Pennsylvania asphalt pavements over three consecutive winters.

	ΔSN_0		
Site	1977-1978	1978-1979	1979-1980
16	28.0	14.0	14.8
17	31.7	24.9	26.4
19	36.3	23.2	19.9
20	27.3	22.4	23.1
21	30.3	21.5	26.2
22	37.8	15.3	32.5

The time constant τ is associated with the rate of decrease in skid resistance over an annual cycle and with the polishing rate of the aggregate. Again, BPN and ADT appear to be useful parameters for prediction. A linear regression of the data, however, yields a poor, though significant, correlation. The resulting relationship for asphalt pavement sites in Pennsylvania is

$$\tau = 56.3 + 0.972\text{BPN} + 0.00721\text{ADT} \quad R = 0.731 \tag{19}$$

The relationship for sites in Tennessee and North Carolina is

$$\tau = -370.1 + 9.008\text{BPN} - 0.0112\text{ADT} \quad R = 0.570 \tag{20}$$

The introduction of polishing parameter BPN_{2000} instead of BPN is found to improve the prediction of τ significantly. For Pennsylvania sites (see Figure 12),

$$\tau = -22.6 + 0.00933\text{ADT} + 2.120\text{BPN}_{2000} \quad R = 0.875 \tag{21}$$

where BPN_{2000} is the measure of the polish susceptibility of the aggregate, the value of BPN after 2,000 cycles of polishing with 0.05-mm silica abrasive and the Penn State reciprocating pavement polisher.

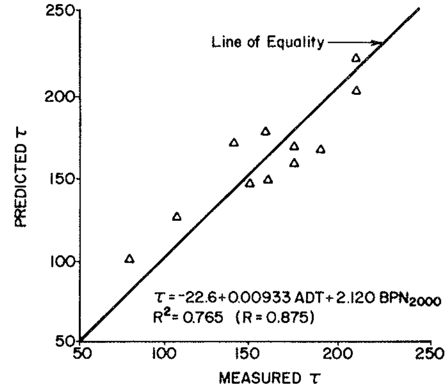
PREDICTION OF SHORT-TERM RESIDUALS

The seasonal variations in skid resistance are assumed to be a function of pavement aggregate properties and traffic density. The short-term residuals, however, are a result of rainfall effects, temperature effects, and errors in skid-resistance measurements. The largest source of measurement errors is the variation in the lateral placement of the test tire. Hill and Henry (17) discussed these three factors on the basis of 1979 data for 21 test pavements in Pennsylvania. A multiple regression of SN_{0R} versus dry spell factor (DSF) and pavement temperature (T_p) was performed. The resulting regression equation was

$$SN_{0R} = 3.79 - 1.17\text{DSF} - 0.104T_p \tag{22}$$

where DSF = ln (t_R + 1), where t_R is the number of days since the last rainfall of 2.5 mm or more. The upper limit is 7 days; hence, $0 \leq t_R \leq 7$. T_p is the pavement temperature at the time of the test, measured continuously in the wheelpath not being tested. The correlation coefficient (R) of this regression was 0.35 (17). The result thus does not yield a good prediction of short-term residuals.

Figure 12. Prediction of τ from ADT and BPN_{2000} for asphalt pavement surfaces: Pennsylvania sites, 1980.



To improve the model, the parameter PNG, which can be deduced from skid-test data by using Equation 1 or predicted from a microtexture measurement (19), was introduced. A multiple regression was performed for the 1980 data. The results for each site are summarized in Table 5. The introduction of PNG was found to improve the prediction of SN_{0R} significantly.

For asphalt pavement surfaces, the regression equation for Pennsylvania (16 sites) is

$$SN_{0R} = -9.971 - 2.654DSF + 0.057T_p + 7.811PNG \quad R = 0.522 \tag{23}$$

The regression equation for Tennessee and North Carolina (8 sites) is

$$SN_{0R} = 3.584 - 0.669DSF - 0.016T_p + 10.022PNG \quad R = 0.539 \tag{24}$$

For PCC surfaces in Pennsylvania, the regression equation is

$$SN_{0R} = -11.464 - 1.049DSF + 0.0005T_p + 10.934PNG \quad R = 0.436 \tag{25}$$

PREDICTION OF ADJUSTED SKID RESISTANCE

Equations 23 and 25 can be used with Equations 8 and 9 to determine the value of SN_{64F} after adjustment for seasonal and short-term effects for the Pennsylvania sites. The models that can be used to predict the level of skid resistance at the end of the year (SN_{64F}) for a measurement taken at any other time during the season (SN_{64}) are as follows. For asphalt pavement surfaces in Pennsylvania,

$$SN_{64F} = SN_{64} - \{\Delta SN_0 \exp[-(t-74)/\tau] - 9.971 - 2.654DSF + 0.057T_p + 7.811PNG\}\exp(-0.64PNG) \tag{26}$$

For PCC surfaces,

$$SN_{64F} = SN_{64} - [(\Delta SN_0/\tau)(\tau - t + 74) - 11.464 - 1.04DSF + 0.0005T_p + 10.934PNG]\exp(-0.64PNG) \tag{27}$$

Figures 13 and 14 show the adjusted SN_{64F} values compared with the original data for two Pennsylvania sites. The results for other sites are similar. Ideally, SN_{64F} should be constant with time after all of the seasonal and short-term effects are accounted for. The comparatively low correlation coefficients obviously limit the ability of regression Equations 23 and 25 to smooth the data for short-term variations.

Table 5. Short-term parameters for Pennsylvania sites: 1980.

Type of Surface	Site	a_1	a_2	a_3	a_4	R^2
Asphalt	1	-11.854	-3.587	0.039	9.807	0.431
	4	-27.665	-3.583	0.116	17.348	0.467
	8	-26.949	-1.171	0.026	24.026	0.665
	9	-24.638	-2.025	0.151	14.447	0.298
	11	-26.006	-2.556	0.084	18.416	0.490
	12	-20.614	-2.658	0.085	17.275	0.538
	13	-20.477	0.086	0.091	15.033	0.283
	15	-18.635	0.725	0.054	15.647	0.229
	16	-16.159	-2.419	-0.042	16.411	0.498
	17	-26.934	-5.594	-0.083	38.053	0.570
	19	-21.138	-2.124	-0.042	21.391	0.506
	20	-23.706	-2.251	-0.019	22.573	0.613
	21	-37.018	-2.785	0.069	32.793	0.877
	22	-43.415	-1.178	0.132	37.166	0.538
	24	-26.206	-2.786	0.013	22.295	0.563
	25	-12.345	-1.277	0.022	11.832	0.109
PCC	2	-29.867	-2.216	0.088	20.154	0.510
	3	-30.446	-0.760	0.124	19.876	0.430
	10	-20.154	-0.493	0.052	13.940	0.178
	14	-15.151	-1.938	0.005	10.821	0.171
	18	-19.174	-0.169	-0.005	23.071	0.567

Note: $SN_{0R} = a_1 + a_2DSF + a_3T_p + a_4PNG$.

Figure 13. Comparison of measured and adjusted SN_{64} for asphalt pavement surface: Pennsylvania site 8, 1980.

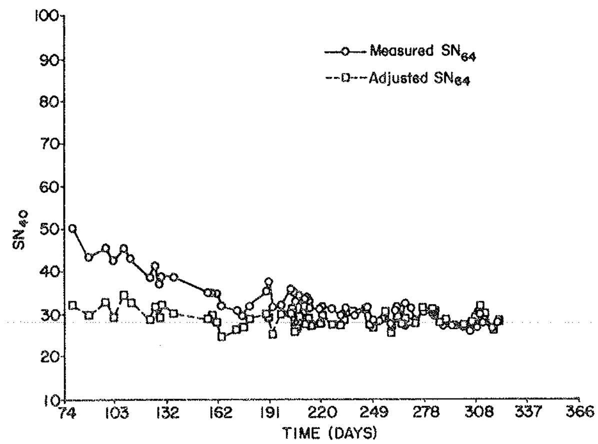
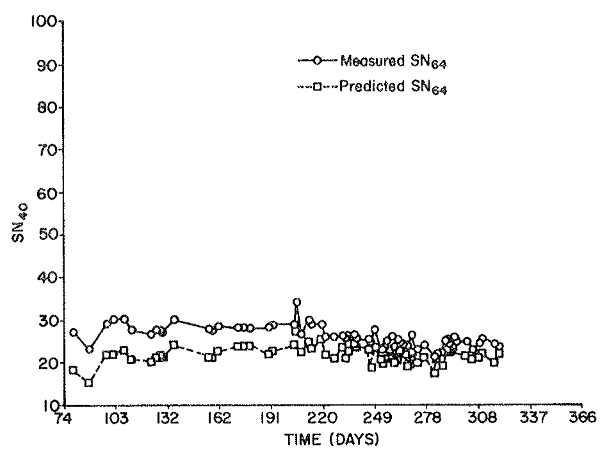


Figure 14. Comparison of measured and adjusted SN_{64} for PCC surface: Pennsylvania site 2, 1980.



The predicted values of SN_{64F} were calculated for each Pennsylvania site by applying Equations 26 and 27 on each day a measurement was made in 1980. The mean and standard deviation of the predictions for each site are compared in Table 6 with the observed values of SN_{64F} for the site. The mean predicted values agree well with the observed values: the standard deviation of the predicted values ranges from 1.68 to 3.70 skid numbers. These values are less than the variations in SN_{64} measurement expected to result from measurement error and other sources of error (20).

It should be noted that the derivation of Equations 26 and 27 requires the assumption that PNG (macrotexture) does not vary between the time of the test and the end of the season. This assumption is reasonable for pavements with relatively durable aggregates and moderate traffic volume, as in the case of the Pennsylvania data. The value of PNG does increase as macrotexture decreases with aggregate wear, which is a function of ADT, aggregate properties, and time.

CONCLUSIONS

The following conclusions can be drawn from the analysis of the mechanistic model:

1. In the course of evaluating the data collected, it was observed that large variations in

Table 6. Comparison of estimated and averaged SN_{64F} values for Pennsylvania sites: 1980.

Site	Observed Values of SN_{64F}	Mean and Standard Deviation of Prediction From All Observations	
		Mean	Standard Deviation
1	26.1	23.4	3.1
2	24.0	22.0	2.2
3	42.4	42.5	3.6
4	27.9	25.8	2.2
8	29.1	29.4	3.0
9	41.8	41.5	3.3
10	47.6	46.5	3.7
11	26.7	25.7	2.5
12	31.3	31.6	2.7
13	55.8	56.3	3.7
14	35.7	33.9	3.2
15	55.0	56.1	3.3
16	19.5	17.1	2.7
17	26.1	25.2	2.8
18	48.0	48.7	2.7
19	26.3	24.7	2.6
20	34.1	33.0	3.2
21	26.1	25.6	1.7
22	46.0	46.6	2.7
24	23.4	21.8	2.6
25	45.1	43.9	2.7

skid-resistance measurements occur systematically over a long period (from one season to another) and over a short period (day to day). The measurements showed that friction levels generally decline from a maximum in early spring to a minimum in late fall and then are rejuvenated to approximately their initial level during the winter. Superimposed on this seasonal cycle are significant short-term variations that result in low skid numbers after a dry period and high (rejuvenated) skid numbers after rainfall.

2. Based on these observations, an effective and simple mechanistic model that treats seasonal and short-term variations separately has been developed. In the model it is hypothesized that seasonal variations are caused by a reduction in the microtexture and the macrotexture as a result of polishing and wear of the aggregate. A procedure for systematically performing this model fitting has also been established.

3. It was found that the level of skid resistance at the beginning of spring is a function of surface microtexture as measured by BPN, average daily traffic volume, and mechanical effects such as the roughening of the surface by studded tires in the winter.

4. The level of SN_0 after removal of the seasonal and short-term effects (SN_{0F}) can be predicted by the average BPN obtained in April and May.

5. The rate of decrease in skid resistance attributable to polishing of the aggregate can be adequately predicted by ADT and by the BPN_{2000} value, which can be obtained by using the Penn State reciprocating pavement polisher.

6. The short-term variations can be predicted by dry spell factor and pavement temperature, but the introduction of PNG is found to improve the short-term prediction model significantly.

7. It has been shown that the mechanistic model developed here can be used to predict the level of skid resistance at the end of the season from a measurement taken at any time during the season.

8. Although fairly good agreement has been found between the results for the sites in Pennsylvania and those in Tennessee and North Carolina, similar investigations should be conducted in other geographic areas.

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REFERENCES

1. C.G. Giles and B.E. Sabey. A Note on the Problem of Seasonal Variation in Skidding Resistance. Proc., 1st International Skid Prevention Conference, Virginia Highway Research Council, Charlottesville, 1959, pp. 563-568.
2. Annual Book of ASTM Standards: Part 15. ASTM, Philadelphia, 1980.
3. M.A. Furbush and K.E. Styers. The Relationship of Skid Resistance to Petrography of Aggregates. Pennsylvania Department of Transportation, Harrisburg, Final Rept., 1972.
4. W.L. Gramling and J.G. Hopkins. Skid Resistance Studies: Aggregate Skid Resistance Relationship as Applied to Pennsylvania Aggregates. Pennsylvania Department of Transportation, Harrisburg, Final Rept., 1974.
5. S.H. Dahir, J.J. Henry, and W.E. Meyer. Seasonal Skid Resistance Variations. Pennsylvania Department of Transportation, Harrisburg, Final Rept., 1979.
6. J.M. Rice. Seasonal Variations in Pavement Skid Resistance. Public Roads, Vol. 40, No. 4, March 1977, pp. 160-166.
7. R.R. Hegmon. Seasonal Variations in Pavement Skid Resistance: Are These Real? Public Roads, Vol. 42, No. 2, Sept. 1978, pp. 55-62.
8. J.J. Henry, K. Saito, and R. Blackburn. Predictor Model for Seasonal Variations in Skid Resistance. FHWA, Final Rept., 1983.
9. J.J. Henry and S.H. Dahir. Predictor Models for Seasonal Variation in Skid Resistance. Pennsylvania Transportation Institute, Pennsylvania State Univ., University Park, Interim Rept. 1, 1979.
10. V.R. Shah and J.J. Henry. The Determination of Skid Resistance: Speed Behavior and Side Force Coefficients of Pavements. TRB, Transportation Research Record 666, 1978, pp. 3-18.
11. Interim Recommendations for the Construction of Skid-Resistant Concrete Pavement. American Concrete Paving Assn., Arlington Heights, Ill., Tech. Bull. 6, 1969, pp. 8-13.
12. S.H. Dahir and W.E. Meyer. Bituminous Pavement Polishing. Pennsylvania Department of Transportation, Harrisburg, Final Rept., 1974.
13. J.J. Henry and S.H. Dahir. Seasonal Skid Resistance Variations. Pennsylvania Department of Transportation, Harrisburg, Interim Rept., Res. Project 75-10, 1978.
14. S.H. Dahir and J.J. Henry. Seasonal and Short-Term Variations in Skid Resistance. TRB, Transportation Research Record 715, 1979, pp. 69-76.
15. J.J. Henry and R.R. Blackburn. Predictor Model for Seasonal Variations in Skid Resistance. Presented at Review Meeting of Federally Coordinated Program of Highway Research Development, Project 1W, Williamsburg, Va., 1979.
16. B.J. Hill and J.J. Henry. Surface Materials and Properties Related to Seasonal Variations in Skid Resistance. In Pavement Surface Characteristics and Materials, ASTM, Philadelphia, Special Tech. Publ. 763, 1982, pp. 76-81.
17. B.J. Hill and J.J. Henry. Short-Term Weather-Related Skid Resistance Variations. TRB, Transportation Research Record 836, 1981, pp. 76-81.

18. R.B. Shakely, J.J. Henry, and R.J. Heinsohn. Effects of Pavement Contaminants on Skid Resistance. TRB, Transportation Research Record 788, 1980, pp. 23-28.
19. M.C. Leu and J.J. Henry. Prediction of Skid Resistance as a Function of Speed from Pavement Texture. TRB, Transportation Research Record 666, 1978, pp. 7-13.
20. W.E. Meyer, R.R. Hegmon, and T.D. Gillespie. Locked-Wheel Pavement Skid Tester Correlation and Calibration Techniques. NCHRP, Rept. 151, 1974.

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Skid-Resistance Measurements with Blank and Ribbed Test Tires and Their Relationship to Pavement Texture

J.J. HENRY AND KAZUO SAITO

A prediction model for the ratio of skid numbers obtained with the ribbed ASTM E501 test tire to those obtained with the blank ASTM E524 test tire at any speed has been developed by using data from 22 pavement test sites in Pennsylvania. The prediction is based on the Penn State model for skid-resistance speed behavior. The model was developed as a function of a macrotexture parameter defined by sand-patch mean texture depth (MTD). Application of this model permits the prediction of the blank-tire skid number at any speed from a measured ribbed-tire skid number and a macrotexture measurement. A simplified model for the blank-tire skid number at a test speed of 64 km/hr was also developed. Values calculated from both models show good agreement with each other as well as with the actual data. An effort was also made to relate skid resistance measured with both types of test tires to pavement texture. The results show a strong relationship between skid numbers with both test tires and pavement macrotexture and microtexture. Therefore, by performing a pavement skid-resistance survey with both the blank E524 and the ribbed E501 test tires, the levels of macrotexture and microtexture can readily be estimated. Seasonal and short-term variations in data with the two tires were also compared. Short-term variations do not pose as great a problem in the blank-tire data as in the ribbed-tire data.

Adequate tire-pavement friction on wet pavement is important for maintaining safe vehicle operation. The wet-pavement friction of the primary highway systems of most states is monitored in annual surveys by using the test procedure specified in ASTM test method E274-79 (1). This method is used to determine the skid resistance of the wet pavement with a ribbed test tire specified by ASTM E501-76 (1) under fully specified test conditions. The E501 test tire has seven smooth, longitudinal ribs separated by six grooves, which provide for drainage of water from the tire-pavement interface as the tire slides over the wet pavement during the test. The specification requires that the tire be discarded when the minimum depth of the grooves reaches 4 mm.

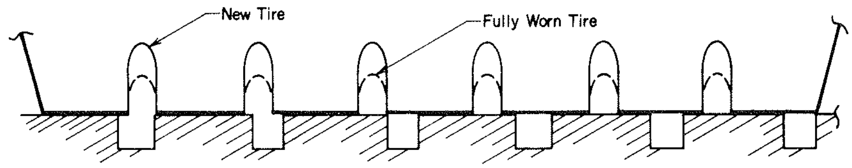
Recently, the use of the E501 test tire for evaluating wet-pavement safety has been questioned (2,3). Pavement grooving is widely accepted as an effective means of reducing skidding accidents on wet pavement. However, the skid number measured with the ribbed tire is not significantly improved by grooving (4,5). In a Michigan study (6), skid-resistance measurements were made with both ribbed and blank tires, both before and after longitudinal grooving, at a site that had a high rate of wet-pavement accidents. Wet-pavement accidents decreased dramatically in the grooved areas, which showed only a slight increase in skid resistance when measured with the ribbed tire but a large increase when measured with the blank tire.

Figure 1 shows a conceptualized profile of a ribbed test tire superimposed on a typical pavement grooving pattern. Because the presence or absence of the grooves does not affect the skid number, it is apparent that sufficient drainage is provided by the tire grooves. Therefore, if the skid number measured with the ribbed test tire were a true measure of safety, pavement grooving could not be justified. Because of its adequate drainage, the ribbed test tire is not sensitive to the drainage capability provided by the pavement macrotexture. The skid resistance measured with the ribbed test tire on dense-graded (fine-textured) pavement would not predict the low friction potential that such a pavement might have for a car with worn tires if the pavement were covered with a thick water film (7).

Several state agencies are investigating the use of the blank tire specified by ASTM E524-76 (1). A Connecticut study suggested that tests with the blank tire correlate well with frequency of wet-pavement accidents, especially hydroplaning accidents, regardless of pavement type (8). A study of 31 test sites in Virginia, including both bituminous and portland cement concrete (PCC) pavements, compared the skid numbers measured with both blank and ribbed tires by grouping the pavements by texture depth (9). On some pavements with high macrotexture the blank and ribbed skid numbers were almost identical, whereas on the pavements with low levels of macrotexture they differed significantly.

A study sponsored by FHWA was initiated at Penn-

Figure 1. Geometry of interface of a ribbed tire and typical grooved pavement (0.65- x 0.65-mm grooves on 25-mm spacing).



sylvania State University to obtain additional data in Pennsylvania and to compare the results with those of the state projects. The preliminary results indicate that the E501 tire is a poor discriminator of macrotexture. Although the safety of pavements can be adequately ranked with a narrow range of macrotexture by using the E501 tire, it is not possible to correctly compare, for example, dense-graded (DG) and open-graded (OG) asphalt pavements or grooved and ungrooved PCC pavements (2). Based on these results, it has been concluded that the ribbed E501 test tire provides a good evaluation of microtexture but is not sensitive to macrotexture, which is a significant factor in wet-pavement safety.

Ideally, a pavement skid-resistance survey should be performed with both the ribbed E501 and the blank E524 tires. By comparing the skid-resistance values obtained from both tires, the levels of microtexture and macrotexture can be readily estimated and thus the cause of poor skid resistance and the choice and likelihood of success of corrective measures can be assessed.

In this paper a prediction model is developed that can be used to estimate the skid-resistance level of pavements for a blank tire from actual measurements made with a ribbed tire and from the pavement macrotexture. An attempt is also made to develop the relationship between pavement texture and skid resistance with both tires.

DATA BASE

Data are available from tests with both blank and ribbed test tires on the 22 pavement test sites of the skid-test program conducted by Pennsylvania State University. The pavements at these sites represent a variety of aggregates and mix designs and include both asphalt and PCC. The pavements are subject to a wide range of average daily traffic. The skid tests were made in the transient slip mode (10), which not only provides SN_{64} data according to ASTM E274 but also yields brake slip numbers at 16, 32, and 48 km/h, which can be used to approximate SN_{16}, SN_{32}, and SN_{48} for both blank and ribbed test tires. Texture measurements made at each site included British pendulum number (BPN) according to ASTM E303 (1) and mean texture depth (MTD) by the sand-patch test according to the American Concrete Paving Association method (11).

ROLE OF PAVEMENT TEXTURE IN SKID RESISTANCE

When skid testing is performed with a particular test tire, pavement surface properties are the main factors that affect the measurement. The pavement surface characteristics that affect skid resistance can be divided into two groups: microtexture and macrotexture. Microtexture, with a space-frequency content greater than 2,000 cycles/m, is a function of the asperities and surface roughness of individual aggregate particles. Macrotexture, with a space-frequency range from 25 to 2,000 cycles/m, is a function of aggregate gradation (12). Microtexture penetrates the water film to provide direct contact with the tire; macrotexture provides channels for water to escape from the tire-pavement interface and thus plays an important role in the prevention of wet-pavement accidents.

Leu and Henry (13) have shown that skid number data decrease exponentially with speed according to the Penn State model:

$$SN_V = SN_0 \exp[-(PNG/100)\,V] \tag{1}$$

where

SN_V = skid number at velocity V (km/h);
SN_0 = skid number/speed intercept; and
PNG = percentage normalized gradient (h/km), defined as $-(100/SN) \cdot [d(SN)/dV]$.

Leu and Henry (13) also found that, for the ribbed-tire test data, SN_0 is highly correlated with such microtexture parameters as height of the microtexture profiles and BPN and that the rate at which the skid number decreases with speed, described by PNG, is correlated with macrotexture parameters such as the height of macrotexture profiles and sand-patch MTD. A significant advantage of this model is that it separates the effects of macrotexture and microtexture. Good skid resistance at traffic speeds such as 64 km/h requires high levels of both macrotexture and microtexture.

MODEL FOR PREDICTION OF SKID NUMBER WITH BLANK TIRE

Relationship Between Skid Number and Speed for Blank and Ribbed Tires

The relationship between skid number and speed for blank and ribbed tires can be developed by using the Penn State model, given as Equation 1. The model for ribbed-tire data can be expressed as

$$SN_V^R = SN_0^R \exp[-(PNG^R/100)\,V] \tag{2}$$

The model for blank-tire data is

$$SN_V^B = SN_0^B \exp[-(PNG^B/100)V] \tag{3}$$

where

SN_V^R = skid number with the ribbed tire at velocity V (km/h),
SN_V^B = skid number with the blank tire at velocity V (km/h),
SN_0^R = skid number/speed intercept for the ribbed tire,
SN_0^B = skid number/speed intercept for the blank tire,
PNG^R = PNG for the ribbed tire, and
PNG^B = PNG for the blank tire.

The ratio of SN_V^B to SN_V^R is then formed:

$$SN_V^B/SN_V^R = (SN_0^B/SN_0^R)\exp[(PNG^R - PNG^B)V/100] \quad (4)$$

or

$$SN_V^B = SN_V^R C_0 \exp(\Delta PNG/100)V \quad (5)$$

where $C_0 = SN_0^B/SN_0^R$ and $\Delta PNG = PNG^R - PNG^B$.

If it is possible to correlate C_0 and ΔPNG with pavement texture, Equation 4 or 5 can be used to predict blank-tire skid number from measured ribbed-tire skid number and pavement texture at any speed. It has been shown in studies in Illinois (14) and New York (15) that the difference between the ribbed-tire and blank-tire skid number is a function of macrotexture and that the differences are larger at low macrotexture than at high macrotexture. Therefore, it is assumed that both C_0 and ΔPNG are the function of macrotexture.

C_0 Versus Macrotexture

To test the hypothesis that a macrotexture parameter can be used to predict C_0, an attempt was made to correlate C_0 with MTD for the data obtained in the fall of 1979 (see Table 1). A high degree of correlation was found, as shown in Figure 2. A least-squares regression analysis yields

$$C_0 = 0.87(MTD)^{0.413} \quad R = 0.958 \quad (6)$$

Here and in subsequent equations, MTD is expressed in millimeters.

ΔPNG Versus Macrotexture

Next, a correlation between ΔPNG and MTD was attempted as a means of testing the hypothesis that ΔPNG can be predicted by macrotexture data. In Figure 3, ΔPNG is plotted versus MTD for the 20 test sites. The resulting relationship is

$$\Delta PNG = 0.0238(MTD)^{-1.75} \quad R = 0.817 \quad (7)$$

The results show that the difference between the ribbed-tire and blank-tire values for PNG decreases sharply as macrotexture increases and approaches zero at high macrotexture.

Prediction of Skid Numbers for Blank Tire from Ribbed-Tire Measurements and Macrotexture

By combining Equations 5 through 7, a relationship among skid number with a blank tire (SN_V^B), skid number with a ribbed tire (SN_V^R), sand-patch MTD, and speed (V) can be obtained:

$$SN_V^B = 0.87\, SN_V^R (MTD)^{0.413} \exp[0.000238V(MTD)^{-1.75}] \quad (8)$$

Skid number values at 32 km/h (SN_{32}^B) and 64 km/h (SN_{64}^B), calculated from the ribbed-tire data at the corresponding speed and macrotexture by using Equation 8, are compared with the measured skid numbers in Figures 4 and 5. Both figures show excellent agreement between measured skid numbers and predicted ones.

Figure 2. C_0 versus MTD: fall 1979.

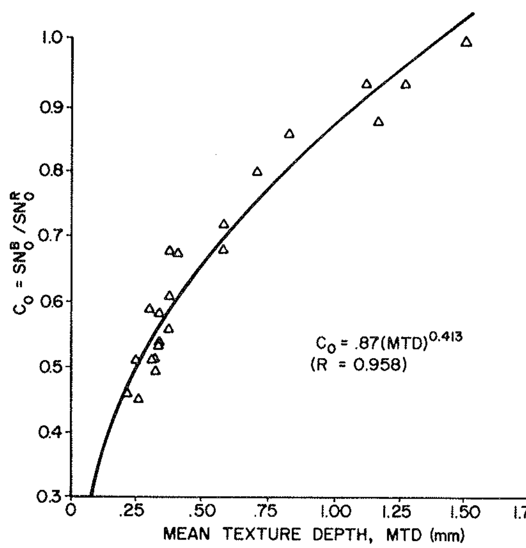


Table 1. Skid-resistance and texture data.

		Summer 1979[a]				Fall 1979[b]		
Site	Type of Pavement	C_{40}	MTD (mm)	$C_0(SN_0^B/SN_0^R)$	ΔPNG (h/km)	C_{40}	MTD (mm)	BPN
1	DG	0.529	0.263	0.451	0.250	0.530	0.263	44.0
2	PCC	0.554	0.300	0.541	0.072	0.557	0.338	58.5
3	PCC	0.527	0.263	0.513	0.163	0.562	0.325	69.0
4	DG	0.642	0.225	0.589	0.078	0.611	0.200	56.5
7	PCC	0.551	0.225	0.512	0.191	0.575	0.250	69.0
8	DG	0.830	0.700	0.801	0.072	0.901	0.700	41.0
9	DG	0.724	0.588	0.722	0.109	0.766	0.575	47.5
10	PCC	0.430	0.225	0.456	0.445	0.527	0.213	65.5
11	DG	0.630	0.263	0.537	0.172	0.597	0.338	51.0
12	DG	0.741	0.438	0.681	0.200	0.769	0.375	57.5
13	OG	0.914	1.025	0.934	0.025	0.939	1.113	87.0
14	PCC	0.515	0.325	0.495	0.272	0.583	0.325	60.5
15	OG	0.996	1.388	0.936	0.034	0.973	1.263	78.0
16	OG	0.671	0.250	0.583	0.166	0.632	0.338	43.0
17	DG	0.878	0.925	0.862	0.044	0.842	0.825	52.5
18	PCC	0.667	0.463	0.674	0.100	0.696	0.400	68.5
19	DG	0.536	0.413	0.562	0.159	0.603	0.375	48.5
20	DG	0.657	0.413	0.610	0.141	0.646	0.375	58.0
21	OG	0.968	1.138	0.876	0.044	0.927	1.163	51.0
22	OG	0.969	1.250	1.001	0.001	0.985	1.488	81.0
24	DG	0.534	0.275	0.508	0.125	0.547	0.313	51.0
25	OG	0.717	0.765	0.681	0.097	0.742	0.575	75.5

[a]Averaged values for July and August. [b]Averaged values for September and October.

Figure 3. ΔPNG versus MTD: fall 1979.

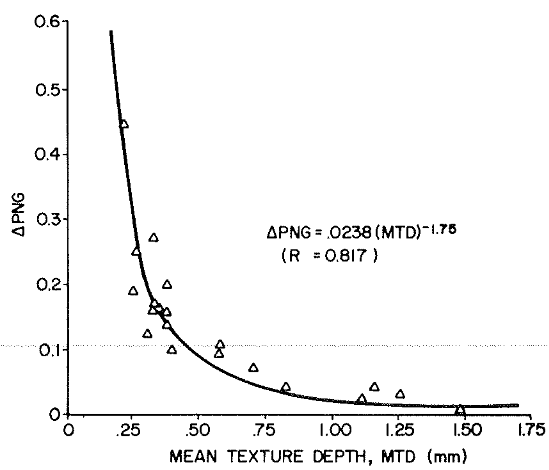
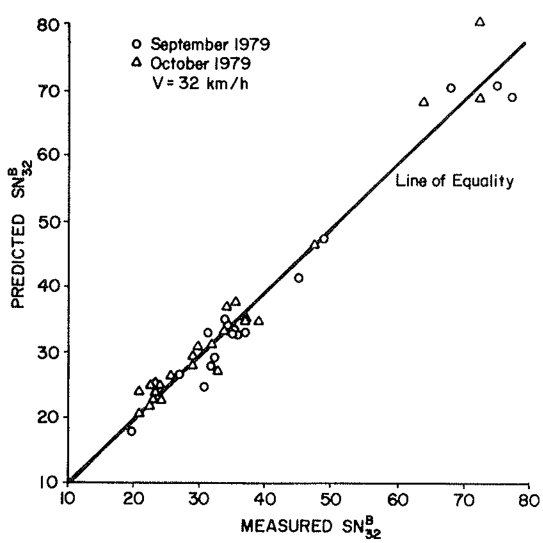


Figure 4. Measured versus predicted SN^B_{32}.



Simplified Model for Testing Speed

The skid test is usually performed at 64 km/h. The model in Equation 8 can be used to predict the skid number with the blank tire at the test speed of 64 km/h, as shown in Figure 5. But, because this model is somewhat complicated, a simplified model was developed.

The skid number measured with the blank tire at 64 km/h is designated SN^B_{64}, and that measured with the ribbed tire is designated SN^R_{64}. The ratio of SN^B_{64} to SN^R_{64}, defined as $C_{64} = SN^B_{64}/SN^R_{64}$, is again correlated with macrotexture (MTD) for the data obtained from the 22 test sites in the summer and fall of 1979 (Table 1). In Figure 6, C_{64} is plotted versus MTD. A least-squares regression analysis yields

$$C_{64} = 0.887(\mathrm{MTD})^{0.36} \quad R = 0.969 \tag{9}$$

or

$$SN^B_{64} = 0.887 SN^R_{64}(\mathrm{MTD})^{0.36} \tag{10}$$

In the same way, an expression for C_{64} can be developed from Equation 8; Figure 6 shows the results of both Equations 8 and 9. Agreement is seen to be

Figure 5. Measured versus predicted SN^B_{64}.

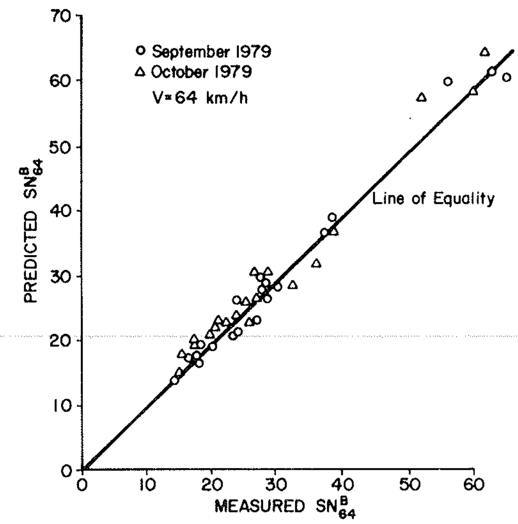
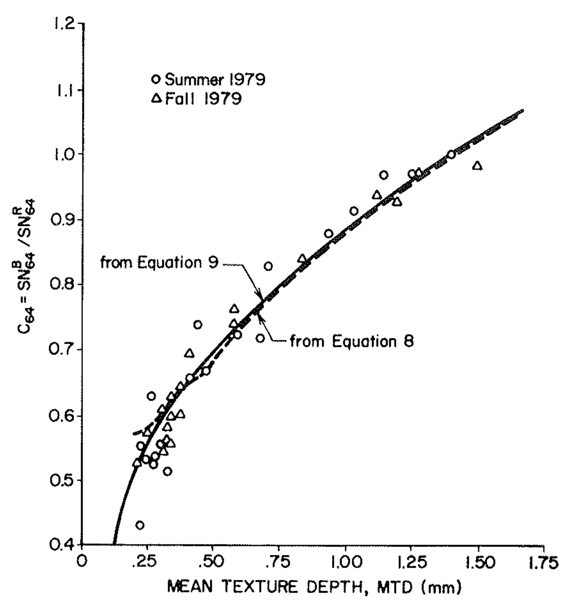


Figure 6. C_{64} versus MTD: summer and fall 1979.



quite good, but Equation 8 must not be used for macrotexture levels below those used in the development of the model--i.e., <0.25 mm.

The relationship between skid numbers with both blank and ribbed tires at 64 km/h for various macrotexture levels is shown in Figure 7. Conversely, it could be shown that the MTD can be predicted by using this relationship when the skid resistance for the pavement with both types of test tires is known.

RELATIONSHIP BETWEEN SKID RESISTANCE WITH BOTH TYPES OF TEST TIRES AND PAVEMENT TEXTURE

Ribbed-Tire Versus Blank-Tire Skid-Test Concept

In an attempt to better define the skid-resistance values of pavements, Henry (2) has compared the skid-resistance data measured with both the ribbed and blank test tires in the fall of 1978 and the spring of 1979. Linear regression equations were used to relate the test results for each tire to a measure of microtexture, defined by BPN measurements, and a measure of macrotexture, defined by MTD as determined from sand-patch tests. The resulting regression equations have shown that the ribbed-tire skid number is highly sensitive to surface microtexture and the blank-tire skid number is sensitive to both macrotexture and microtexture. The expressions for BPN and MTD were preliminary at that time and required further validation. However, the concept of using both types of skid-test data shows promise as an indirect method of macrotexture and microtexture measurement.

Correlation of Skid Numbers with Texture Data

Data are available from skid tests conducted with both types of tires during April and October in 1979 and 1980 on 22 test sites in Pennsylvania. As in the previous study, linear regression equations were used to correlate the test results for each tire with BPN and MTD. The multiple regression analysis was performed on all data in the following form:

$$SN_{64}^{R} = a_0 + a_1 MTD + a_2 BPN \quad (11)$$

$$SN_{64}^{B} = b_0 + b_1 MTD + b_2 BPN \quad (12)$$

The resulting regression equations are

$$SN_{64}^{R} = -9.7 + 4.72MTD + 0.766BPN \quad R = 0.922 \quad (13)$$

$$SN_{64}^{B} = -19.5 + 17.3MTD + 0.628BPN \quad R = 0.917 \quad (14)$$

The coefficients are quite similar to the earlier results (2). They confirm the conclusion that skid measurements with the ribbed test tire are highly sensitive to pavement surface microtexture and relatively insensitive to macrotexture and that skid measurements with the blank test tire are sensitive to both macrotexture and microtexture.

Correlation of Texture Data to Skid Numbers

Equations 13 and 14 could be solved for BPN and MTD in terms of both SN_{64}^{R} and SN_{64}^{B}. However, to examine the validity of the correlation, a linear regression analysis of the data was performed to relate BPN and MTD to skid numbers with both tires. The multiple regression analysis was performed on all data in the following form:

$$BPN = c_0 + c_1 SN_{64}^{R} + c_2 SN_{64}^{R} \quad (15)$$

$$MTD = d_0 + d_1 SN_{64}^{R} + d_2 SN_{64}^{B} \quad (16)$$

The resulting regression equations are

$$BPN = 20.0 + 0.405SN_{64}^{R} + 0.039SN_{64}^{B} \quad R = 0.905 \quad (17)$$

$$MTD = 0.490 - 0.0289SN_{64}^{R} + 0.0426SN_{64}^{B} \quad R = 0.853 \quad (18)$$

As expected, the result for BPN shows that SN_{64}^{B} plays only a small role in the prediction of the level of BPN and it may be possible to predict BPN solely from SN_{64}^{R}. To test this hypothesis, an attempt was made to correlate BPN with SN_{64}^{R} for all available data. The least-squares analysis yields

$$BPN = 22.2 + 0.998SN_{64}^{R} \quad R = 0.894 \quad (19)$$

COMPARISON OF SEASONAL VARIATIONS IN SKID RESISTANCE WITH RIBBED AND BLANK TIRES

Skid-resistance measurements with the ribbed tire on public highways in Pennsylvania and other states have exhibited seasonal and short-term variations (16,17). Extreme seasonal variations as high as 30 skid numbers have been observed along with more typical variations in the range of 5 to 15 (17). These variations make it difficult to establish a rational maintenance program in which skid resistance is an important factor.

Data are available from tests with the ribbed and blank tires for 1980. Figures 8 and 9 compare seasonal variations in skid number (SN_{64}) with the ribbed and blank tires for a dense-graded asphalt surface (site 1) and for a PCC surface (site 4). The figures show clearly that long-term (seasonal) variations for both tires exhibit almost the same trend whereas short-term (daily) variations for both tires are significantly different.

As shown in the paper by Saito and Henry in this Record, the short-term variations in skid resistance with the ribbed tire show fairly large fluctuations resulting from rainfall, pavement temperature, and short-term changes of microtexture parameters and PNG. On the other hand, the short-term fluctuations with the blank tire are small and probably negligible. The standard deviations of the skid numbers with the ribbed tire are 1.83 for the asphalt surface and 2.05 for the PCC surface; the corresponding

Figure 7. Relationship of SN_{64}^{R}, SN_{64}^{B}, and MTD.

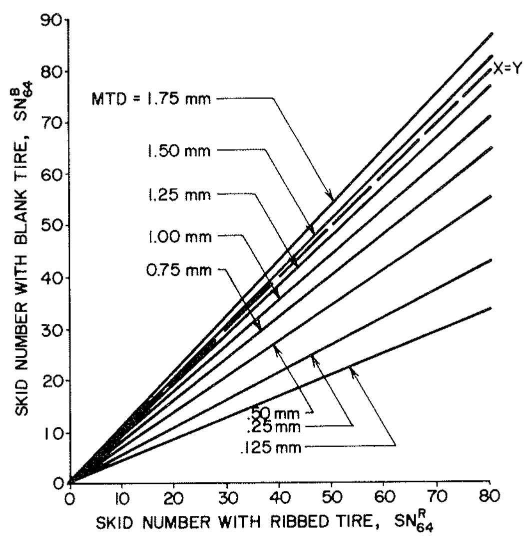


Figure 8. Comparison of seasonal variations in SN_{64} with ribbed and blank tires: PCC surface, 1980.

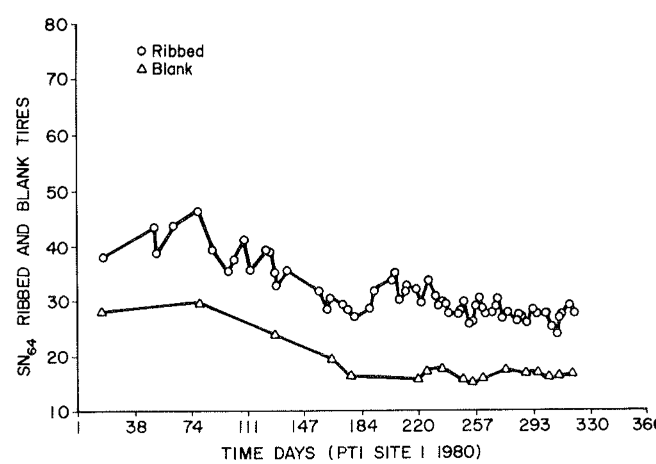
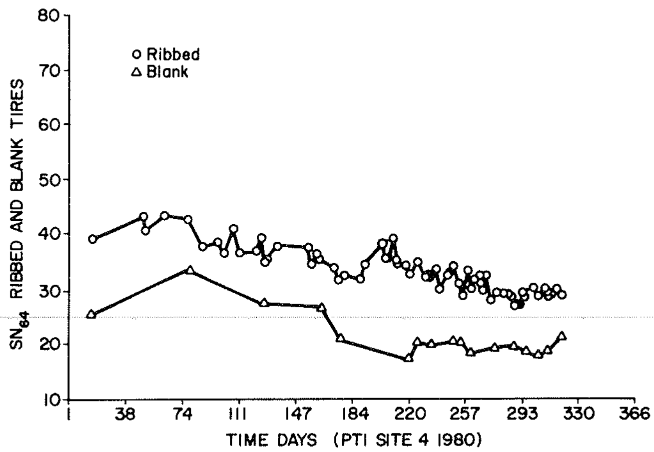


Figure 9. Comparison of seasonal variations in SN_{64} with ribbed and blank tires: asphalt surface, 1980.



standard deviations with the blank tire are 0.65 and 0.91.

It can be concluded from these results that the measurements with the blank test tire are not sensitive to short-term variations and therefore that the blank E524 tire is less of a problem with respect to short-term variations in skid-resistance measurements.

CONCLUSIONS AND RECOMMENDATIONS

In this study, the prediction model for the ratio of skid resistance with a ribbed tire to that with a blank tire at any speed has been developed by using the Penn State model for skid-resistance/speed behavior. The model was developed as a function only of a macrotexture parameter, described by sand-patch MTD. By using this model, skid-resistance levels can be predicted for the blank tire at any speed from a measured skid number obtained with the ribbed tire at the same speed and a macrotexture measurement. For the user's convenience, a simplified model has been developed to predict skid number with the blank tire at a test speed of 64 km/h. The values calculated from both models show good agreement.

The study results also show that the ribbed E501 tire provides a good evaluation of microtexture but is not sensitive to macrotexture, which is an important factor in wet-pavement safety. The blank E524 tire is sensitive to both macrotexture and microtexture. If both macrotexture and microtexture measurements are made, the level of skid resistance can readily be estimated. Conversely, by performing a pavement skid-resistance survey with both the ribbed and blank tires, the level of pavement microtexture and macrotexture can be estimated.

Because skid-test trailers are extensively used by most states, this concept of indirect texture measurement can be implemented easily and relatively inexpensively. If skid-resistance surveys are to be performed with only one type of tire, the blank E524 tire appears to be the stronger candidate, because it is more sensitive to macrotexture and poses less of a problem with respect to short-term variations in skid resistance.

ACKNOWLEDGMENT

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REFERENCES

1. Annual Book of ASTM Standards: Part 15. ASTM, Philadelphia, 1980.
2. J.J. Henry. Use of Blank and Ribbed Test Tires for Evaluating Wet Pavement Friction. TRB, Transportation Research Record 788, 1981, pp. 1-6.
3. Safety Effectiveness Evaluation: Selected State Highway Skid Resistance Programs. National Transportation Safety Board, Rept. NTSB-SEE-80-6, Sept. 1980.
4. R.J. Rasmussen. Pavement Surface Texturing and Restoration for Highway Safety. Presented at 53rd Annual Meeting, HRB, 1974.
5. V.R. Shah and J.J. Henry. Relationship of Locked Wheel Friction to That of Other Test Modes. Pennsylvania Department of Transportation, Harrisburg, Final Rept., Feb. 1977.
6. F. Copple and P.T. Luce. Determination and Improvement of Relevant Pavement Skid Coefficients. Michigan Department of State Highways and Transportation, Res. Rept. R-1038, Jan. 1977.
7. R.R. Hegmon. Wet Weather Accidents and Pavement Skid Resistance. Public Roads, Vol. 45, 1981, pp. 68-74.
8. G.A. Ganung and F.J. Kos. Wet-Weather High-Hazard Locations Identification and Evaluation. Connecticut Department of Transportation, Wethersfield, Final Rept. 403-F-79-4, May 1979.
9. D.C. Mahone. An Evaluation of the Effects of Tread Depth, Pavement Texture, and Water Film Thickness on Skid Number-Speed Gradients. Virginia Highway and Transportation Research Council, Charlottesville, Rept. VHTRC-75-R40, March 1975.
10. V.R. Shah and J.J. Henry. The Determination of Skid Resistance-Speed Behavior and Side Force Coefficients of Pavements. TRB, Transportation Research Record 666, 1978, pp. 13-18.
11. Interim Recommendation for the Construction of Skid-Resistant Concrete Pavement. American Concrete Paving Assn., Arlington Heights, Ill., Tech. Bull. 6, 1969.
12. J.J. Henry. The Relationship Between Texture and Pavement Friction. Presented at 1977 Kummer Lecture, ASTM Annual Meeting, St. Louis, Mo., Dec. 1977.
13. M.C. Leu and J.J. Henry. Prediction of Skid Resistance as a Function of Speed from Pavement Texture. TRB, Transportation Research Record 666, 1978, pp. 7-13.
14. P.G. Dierstein. Tread Tire Versus Smooth Tire Friction on Various Portland Cement Concrete and Bituminous Surface Texture in Illinois. Presented at 1979 Research Review Conference of the Federally Coordinated Program of Highway Research and Development, Williamsburg, Va., Dec. 1979.
15. J.E. Grady and N.P. Chamberlin. Groove-Depth Requirements for Tire-Textured Pavements. New York State Department of Transportation, Albany, Res. Rept. 86, June 1981.
16. S.H. Dahir, J.J. Henry, and W.E. Meyer. Seasonal Skid Resistance Variations. Pennsylvania Department of Transportation, Harrisburg, Final Rept., 1979.
17. J.M. Rice. Seasonal Variations in Pavement Skid Resistance. Public Roads, Vol. 40, No. 4, March 1977, pp. 160-166.

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Tire Noise and Its Relation to Pavement Friction

ROBERT G. BARAN AND J.J. HENRY

Pavement friction and near-field and far-field tire-pavement noise were measured on six pavements representing a wide range of textures. Tire-pavement friction was measured in full-scale locked-wheel tests with both blank and ribbed test tires, and British pendulum numbers were measured. Near-field tire noise was measured on the same pavements by an on-board microphone mounted near the tire. Far-field tire noise was measured by coasting the vehicle through the test sites past a microphone located 15 m from the edge of the pavement. Attempts were made to correlate near-field tire noise with pavement texture to determine whether characteristics of the near-field noise spectrum could be used to predict texture and skid resistance. The data from portland cement concrete pavements produced satisfactory results, but the correlation was poor for asphalt concrete pavements. Relationships among the three measures of pavement skid resistance and far-field noise were developed. Good correlations were obtained between pavement friction and A-weighted sound pressure levels at 64 and 80 km/h, but at 48 km/h the correlations were poor.

The noise generated at the interface between a tire and the surface over which it operates is a function of speed, tire properties, and tire operating conditions. Other factors, such as surface temperature and weather conditions, may also play a role but probably a secondary one. For a given tire operating under fixed conditions, the differences between the noise generated on two pavements must be attributed to differences in the texture of the pavements.

Skid resistance, or the friction of a test tire sliding over a wet pavement, is a function of the same factors. The skid resistance of a pavement can be measured in accordance with ASTM test methods E274 and E303 (1). In this research the full-scale locked-wheel skid resistance of the pavements was measured with both a blank test tire (ASTM E524) and a ribbed test tire (ASTM E501) (1). The values of skid resistance measured with the two tires, together with the British pendulum number (BPN) (ASTM E303), were used to characterize the frictional performance of the pavements.

The first objective of this study was to use the near-field tire noise signature (the characteristics of the noise spectrum generated near the tire-pavement interface and sensed with an on-board microphone) to predict the skid resistance of the pavement. If successful, this would provide an alternative method for measuring skid resistance. The second objective was to determine whether there is a strong relationship between skid resistance and the A-weighted sound levels (dB_A) of far-field tire noise (the sound pressure level of the noise received by a microphone placed 15 m from the roadway as a vehicle coasts through a 30-m length of the roadway). The skid resistance was measured over the same 30-m path. Attempts were made to relate the A-weighted sound pressure levels of the far-field noise to the three measures of pavement friction: BPN, blank-tire skid resistance (SN^B_{64}), and ribbed-tire skid resistance (SN^R_{64}).

EXPERIMENTAL APPROACH

Considerable work has been done on identifying the major sources of noise generated by a moving vehicle. Veres (2) and Eaton (3) have categorized these sources as follows: the tire-pavement interface, aerodynamic sources, engine, and exhaust. Engine and exhaust noise can be eliminated by coasting the vehicle with the engine off. The aerodynamic sources contribute little to noise at higher frequencies, as shown by Hayden (4). A study done at the General Motors Proving Ground by Richards (5) upholds the findings of Hayden.

If these three noise sources are eliminated, tire-pavement noise becomes the dominant source. Tire-pavement noise is affected by seven parameters (2-4,6): vehicle speed, wheel load, inflation pressure, tread pattern, degree of tread wear, tire size and construction, and road surface texture. To investigate the relationship of tire-pavement noise to texture, the first six parameters must be held constant. The variations in the resulting tire noise are then caused solely by pavement surface texture and may be useful as measures of it.

DATA COLLECTION

Near-Field Noise Tests

Recordings of near-field tire-pavement noise have been made by a number of researchers (2-5,7). The method used in this study is similar to that used by Veres (2) and Eaton (3), which was developed at Pennsylvania State University in 1972 (see Figure 1).

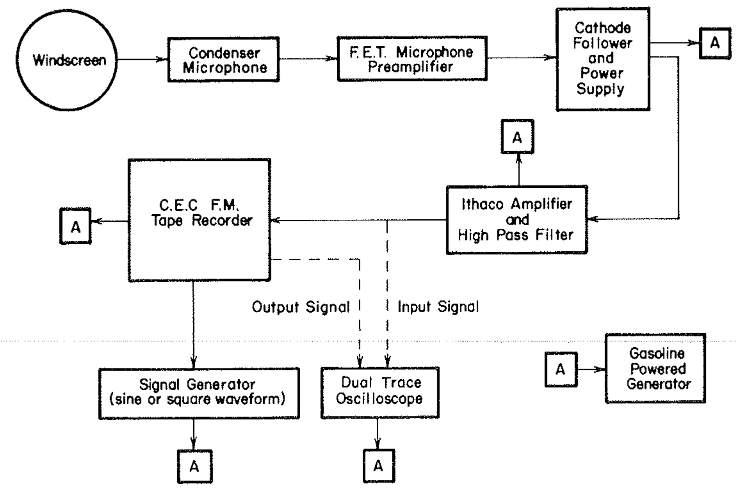
The testing was conducted at 22 sites on public roads where the Pennsylvania Transportation Institute (PTI) has performed frequent measurements of texture and skid resistance. A skid tester was used to record the noise. The test wheel was stripped of all unnecessary equipment to reduce extraneous noise. The microphone was mounted 53 cm behind the axis of the test tire and 20 cm above the road surface. This position was determined by Veres and Eaton to be the optimum one to maximize the response to texture changes. Each component was calibrated by using standard procedures recommended by the manufacturers. The system was calibrated by using a 250-Hz, 124-dB piston phone with an output of 1 V peak-to-peak. The amplifier gain was then set so that the maximum noise level was also 1 V peak-to-peak at 64 km/h.

The tape recorder was operated in the frequency modulation mode, which yielded a bandwidth from 0 to 10,000 Hz, and the high-pass filter was set at 100 Hz. Therefore, the overall effective bandwidth was 100 to 10,000 Hz. A signal generator was used to locate each site on the tape. The motor generator used to power the system was isolated so that it would not contribute to the noise. The testing was conducted by coasting the vehicle at 64 km/h during the late night hours to eliminate any pass-by noise from other vehicles on the roads. Before and immediately after each test, the piston phone was used to check the system to ensure that it was operating properly and remained in calibration.

Far-Field Noise Tests

Thus far, only limited success has been noted in attempts to relate far-field tire noise to texture data. Some positive results have been reported for pavements with large differences in texture depth (8-10). By coasting the vehicle noise from the engine and exhaust can be eliminated, but many site-related problems remain, such as reflecting surfaces, ground absorption, thermal gradients, wind speeds greater than 20 km/h, ambient sound levels, and the presence of other vehicles.

Figure 1. System for measurement of near-field tire-pavement noise.



To minimize these site-related problems, additional tests were conducted at the PTI test track. The six sites to be tested were chosen adjacent to each other. The same recording equipment was used for both near-field and far-field noise. The method used to record the far-field noise was a combination of two SAE methods: SAE J57a, Sound Level of Highway Truck Tires, and SAE J986b, Sound Level for Passenger Cars and Light Trucks. The test site configuration, which was as described in SAE J57a, is shown in Figure 2. Several runs were made at speeds of 48, 64, and 80 km/h.

DATA ANALYSIS

Digitization

The analog noise signals were digitized at the Pennsylvania State University Hybrid Computer Center. The EAI 680 analog patch panel uses an internal clock to sample the analog signal at a rate of 20 kHz. The digitized signal was stored in memory in blocks of 1024 points. The memory was then transferred to magnetic tape for analysis. Each site consisted of at least 48 blocks, or 49,152 points, corresponding to 2.625 sec of real sampling time. To eliminate any transients and to analyze the steady-state response, a start-up time of 2 sec of real time was allowed at the beginning of each site. The block diagram in Figure 3 shows how this was achieved.

The analog signal was recorded at a tape speed of 152 cm/sec. The tape was played back at a speed of 19 cm/sec to maximize the frequency response in the 1- to 10-kHz range, which was expected to be of greatest interest. Figures 4 and 5 show typical power spectral densities in decibels versus frequency for the near-field noise tests.

Statistical Analysis

The Statistical Analysis System (SAS) package was used for all statistical analyses. The stepwise procedure was the basis of the work, and maximum correlation coefficient (R) was used as the main criterion for model improvement. This procedure looks for the best single-variable model, the best two-variable model, and so forth. It begins by finding the single-variable model with the highest correlation, then adds or deletes the next highest variable to produce the greatest improvement in correlation. Also included was a technique that en-

Figure 2. Site for tests of far-field noise.

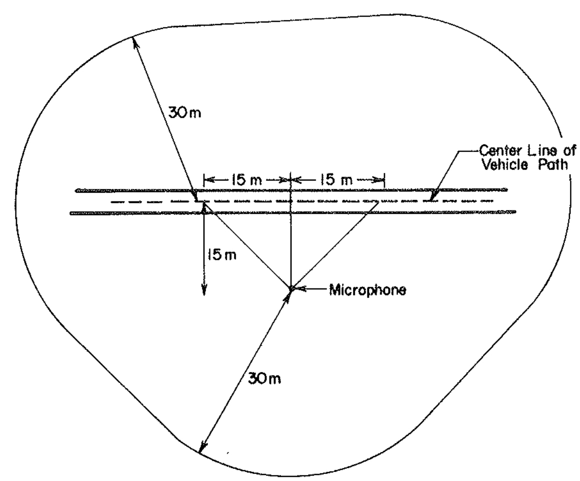
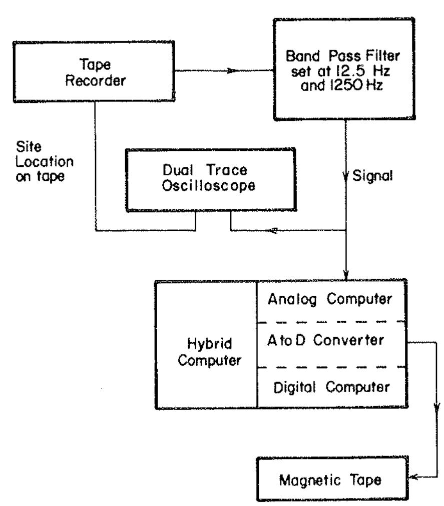


Figure 3. Block diagram of digitization procedure.



tered a variable only if that variable maintained a significance level greater than 90 percent. After the final model was formulated, the correlations among the variables were determined.

Near-Field Noise Spectrum Versus Skid-Resistance Data

An inclusive approach was used. The noise spectrum was regressed against all of the available measurement techniques, including 64-km/h skid-resistance measurements with the blank tire (SN^B_{64}), the ribbed tire (SN^R_{64}), and BPN. These regressions were done for each site, and 512 points of data were used for each variable. Figure 6 shows that the spectrum has three humps--one from 10 to 2.7 kHz, a second from 2.7 to 7 kHz, and a third from 7 to 10 kHz. Therefore, the spectrum was divided into three sections, and regressions that used all of the variables were applied to see which, if any, produced satisfactory correlations and which section of the spectrum had the highest level of significance in predicting pavement friction.

The site-by-site analysis showed that SN_{64} (blank and ribbed tires) and BPN had the highest correlations with the midsection of the spectrum. This result suggested dividing the spectrum into 10 bandwidths, as defined in Table 1. Sets of regression equations were developed for asphalt and portland cement concrete (PCC) sites:

$$a_1SL1 + a_2SL2 + a_3SL3 + a_4SL4 + a_5SL5 + a_6SL6 + a_7SL7 + a_8SL8 + a_9SL9 + a_{10}SL10 + a_{11} = SN^B_{64} \qquad (1)$$

$$b_1SL1 + b_2SL2 + b_3SL3 + b_4SL4 + b_5SL5 + b_6SL6 + b_7SL7 + b_8SL8 + b_9SL9 + b_{10}SL10 + b_{11} = SN^R_{64} \qquad (2)$$

$$c_1SL1 + c_2SL2 + c_3SL3 + c_4SL4 + c_5SL5 + c_6SL6 + c_7SL7 + c_8SL8 + c_9SL9 + c_{10}SL10 + c_{11} = BPN \qquad (3)$$

where SL1 to SL10 are frequency bands defined in Table 1.

The preceding approach determined the best weighting of the spectra to find a model that could be used for either asphalt concrete or PCC surfaces. The spectra consisted of 512 points, and the midsection from 2,700 to 7,000 Hz contained approximately 170 points; a three-point average for each bandwidth was used. The three models were developed from the averaged points for each site to relate the weighted noise levels to the skid-resistance data.

Far-Field Noise Levels Versus Skid Resistance Data

An overall approach was again used to determine the relationship between the noise spectra and the friction data variables. To reduce the number of data points, a smoothing routine was applied. This yielded a spectrum with 128 points, which was then reduced to the A-weighted sound pressure level. Figure 7 shows the power spectral density before the smoothing, and Figure 8 shows the same spectrum after the smoothing. The far-field noise levels are given in Table 2.

To simulate this model the following equations were used:

$$dB_A = aSN^B_{40} + b \qquad (4)$$

$$dB_A = aSN^R_{40} + b \qquad (5)$$

$$dB_A = aBPN + b \qquad (6)$$

$$dB_A = d_1SN^B_{64} + d_2SN^R_{64} + d_3BPN + d_4 \qquad (7)$$

These regressions were performed for each of the six sites for noise levels processed at speeds of 48, 60, and 80 km/h. Another test was used to verify that the topography of the PTI test track met SAE standards. A dense-graded asphalt site was located that complies with the topography required by SAE and has a surface similar to that of the dense-graded asphalt site at the PTI test track. A comparison of the spectra of these two sites showed close agreement. A regression to compare the spectra for these two sites, characterized by 72 points each for three test speeds, yielded the following result:

Figure 4. Power spectral density of near-field noise versus frequency for a PCC pavement.

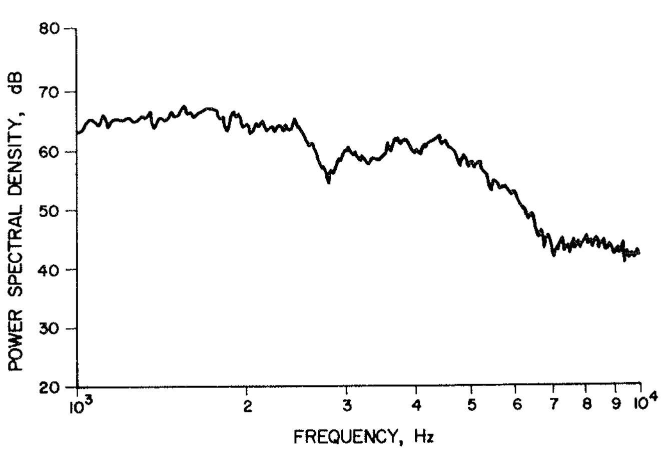


Figure 5. Power spectral density of near-field noise versus frequency for an asphalt pavement.

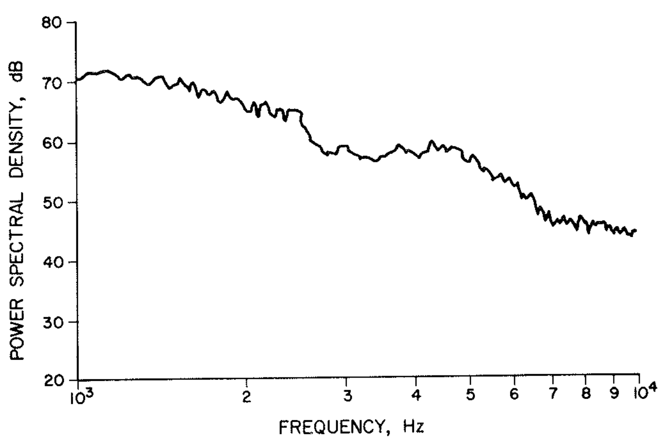


Figure 6. Location of three humps in near-field noise spectrum.

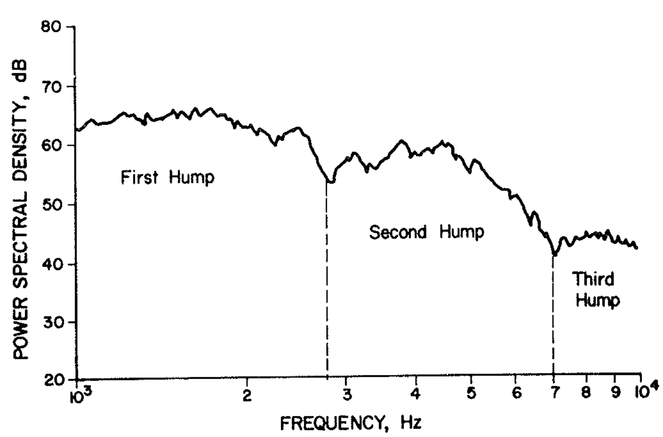


Figure 7. Power spectral density of far-field noise before smoothing.

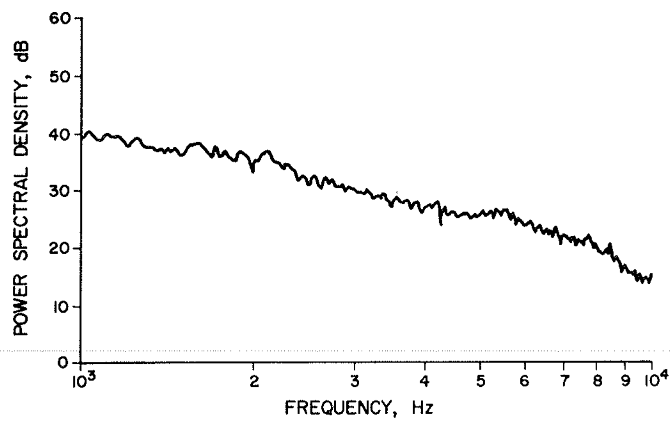
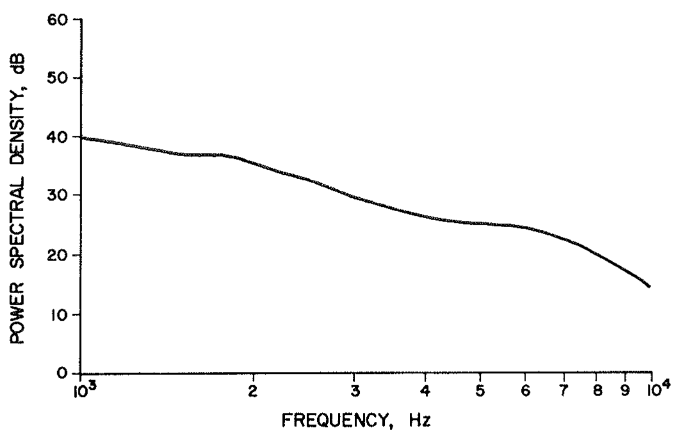


Figure 8. Power spectral density of far-field noise after three-point smoothing routine was used.



$$dB(\text{test track}) = 0.998\ dB(SAE) + 1 \times 10^{-7} \approx dB(SAE) \qquad R = 0.99 \tag{8}$$

This result showed that there was good agreement between the two sites and that the site geometry at the test track was satisfactory in the range of noise frequencies tested.

RESULTS

Near-Field Noise

Correlation at the 90 percent confidence level has been demonstrated between tire-pavement noise spectral density and pavement friction measurements (3). The investigation of the 10 spectral bands between 2.7 and 10 kHz produced satisfactory correlations and reasonable regression equations for PCC pavements. The following equations were obtained:

$$\begin{aligned}(-0.4337)SL1 &+ (-0.3497)SL2 + (-0.5061)SL3 + (1.1899)SL4 \\ &+ (0.3473)SL7 + (-0.7441)SL8 + (-0.3397)SL9 \\ &+ (0.6721)SL10 + 31.14 = SN_{64}^{B}\end{aligned} \tag{9}$$

$$\begin{aligned}(-0.5553)SL1 &+ (-0.4785)SL2 + (-1.1876)SL3 + (2.0443)SL4 \\ &+ (-0.9524)SL8 + (0.4481)SL9 + (1.3398)SL10 \\ &+ 58.89 = SN_{64}^{R} \qquad R = 0.86\end{aligned} \tag{10}$$

$$\begin{aligned}(-0.5378)SL2 &+ (-1.357)SL3 + (1.960)SL4 + (-0.5995)SL6 \\ &+ (-0.9549)SL7 + (1.4345)SL10 \\ &+ 84.63 = BPN \qquad R = 0.72\end{aligned} \tag{11}$$

Table 1. Bounds for central hump of near-field noise spectrum.

	Frequency Range (kHz)		
Band Number	Lower Bound	Upper Bound	Center Frequency
SL1	2.7	3.13	2.915
SL2	3.13	3.56	3.345
SL3	3.56	3.99	3.775
SL4	3.99	4.42	4.205
SL5	4.42	4.85	4.635
SL6	4.85	5.28	5.065
SL7	5.28	5.71	5.495
SL8	5.71	6.14	5.925
SL9	6.14	6.57	6.355
SL10	6.57	7.0	6.785

Table 2. Far-field tire noise data.

Site	$dB_{A_{48}}$	$dB_{A_{64}}$	$dB_{A_{64}}$	SN_{64}^{B}	SN_{64}^{B}	BPN
3	18.88	40.82	47.64	23.2	31.2	61.4
4	37.67	44.20	53.40	22.8	30.6	59.3
5	36.49	51.84	50.52	26.9	44.1	72.5
6	35.10	41.36	42.75	25.2	30.9	65.7
7	–	44.20	45.68	25.9	44.3	74.1
8	38.04	44.87	46.97	23.6	33.6	64.8

The results for the asphalt pavements, obtained by using the same criteria, were less satisfactory:

$$\begin{aligned}(0.6078)SL1 &+ (-0.3445)SL3 + (0.2955)SL4 + (-0.8380)SL7 \\ &+ (1.1523)SL8 + (-0.7789)SL9 \\ &+ 7.886 = SN_{64}^{B} \qquad R = 0.51\end{aligned} \tag{12}$$

$$\begin{aligned}(-0.6455)SL1 &+ (0.3120)SL2 + (-0.8579)SL6 + (0.9286)SL7 \\ &+ (1.0147)SL9 + (-0.6905)SL10 \\ &+ 38.723 = SN_{64}^{R} \qquad R = 0.45\end{aligned} \tag{13}$$

$$\begin{aligned}(-0.7086)SL1 &+ (0.4217)SL2 + (-0.2564)SL4 + (-0.6406)SL6 \\ &+ (0.9340)SL7 + (0.9921)SL9 + (-0.6856)SL10 \\ &+ 39.536 = BPN \qquad R = 0.46\end{aligned} \tag{14}$$

In all cases, regression analysis with parabolic and logarithmic variables yielded poorer results than did the linear regression.

Far-Field Noise

The single-variable regressions in the form of Equations 4, 5, and 6 yielded poor results: R ranged from 0.077 to 0.69. Of these, the regressions with the 64-km/h data and correlation coefficients ranging from 0.58 to 0.69 were the best.

Two-variable models were investigated by using the two skid numbers as the independent variables. As in the case of the single-variable models, the most significant result was obtained for the 64-km/h data:

$$dB_{A_{64}} = 0.88\ SN_{64}^{B} + 0.30\ SN_{64}^{R} + 11.66 \qquad R = 0.71 \tag{15}$$

The three-variable regressions in the form of Equation 7 yielded relatively high correlation coefficients at 64 and 80 km/h but poor correlations at 48 km/h:

$$dB_{A_{48}} = 2.03\ SN_{64}^{B} - 0.007\ SN_{64}^{R} - 0.039\ BPN - 14.27 \qquad R = 0.38 \tag{16}$$

$$dB_{A_{64}} = 3.53\ SN_{64}^{B} + 1.10\ SN_{64}^{R} - 1.62\ BPN + 25.61 \qquad R = 0.85 \tag{17}$$

$$dB_{A_{80}} = 2.07\ SN_{64}^{B} + 1.54\ SN_{64}^{R} - 2.30\ BPN + 93.61 \qquad R = 0.93 \tag{18}$$

CONCLUSIONS

It has been shown that there are significant relationships between tire-pavement friction and tire-pavement noise. Near-field noise data obtained by the procedures developed in this research have spectral characteristics related to the skid resistance of PCC pavements. Over the wider range of asphalt concrete surfaces included in this study, however, poor agreement was noted between the spectral characteristics and pavement friction.

Far-field noise data at 64 and 80 km/h show definite relationships with skid resistance. The most significant variable in these relationships is blank-tire skid resistance. This finding indicates the important effect of pavement macrotexture on far-field tire noise in that blank-tire skid resistance is most strongly affected by pavement macrotexture (11). Increasing skid resistance and increasing macrotexture are seen to produce increased levels of far-field noise.

ACKNOWLEDGMENT

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REFERENCES

1. Annual Book of ASTM Standards: Part 15. ASTM, Philadelphia, 1980.
2. R.E. Veres. Pavement Macrotexture Characterization by Tire Noise. Pennsylvania State Univ., University Park, Automotive Research Program Rept. S62, 1974.
3. A.L. Eaton. Tire Sound: Its Relationship with Surface Macrotexture and Skid Resistance. Pennsylvania State Univ., University Park, Automotive Research Program Rept. S73, 1976.
4. R.E. Hayden. Roadside Noise from the Interaction of a Rolling Tire with the Road Surface. Presented at 81st Meeting of the Acoustical Society of America, Washington, D.C., April 1971.
5. M.G. Richards. Automotive Tire Noise: A Comprehensive Study. Sound and Vibration, May 1973.
6. R.K. Hillquist and P.C. Carpenter. A Basic Study of Automobile Tire Noise. Sound and Vibration, Feb. 1974.
7. T.D. Siddor. Noise Generation Mechanisms for Passenger Car Tires. Presented at 84th Meeting of the Acoustical Society of America, Miami, Fla., Dec. 1973.
8. R.J. Mitrey, D.E. Amsler, and D.E. Surronen. Effects of Selected Pavement Surface Textures on Tire Noise. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Rept. 28, 1975.
9. U. Sandberg. Road Texture Induced External Tire Noise: Empirical Frequency Response Function for Tires. National Swedish Road and Traffic Research Institute, Linköping, Rept. 174A, 1979.
10. J.C. Walker and D.J. Major. Noise Generated at the Tyre Road Interface. In Stress Vibration and Noise Analysis in Vehicles, Applied Science Publishers, Essex, England, 1975.
11. J.J. Henry. The Use of Blank and Ribbed Test Tires for Evaluating Wet Pavement Friction. TRB, Transportation Research Record 788, 1981, pp. 1-5.

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Pavement Edges and Vehicle Stability: A Basis for Maintenance Guidelines

RICHARD A. ZIMMER AND DON L. IVEY

The repair of pavement edge geometry adjacent to unpaved shoulders is a maintenance activity that requires continuous effort on the part of all state and local agencies. Although it is generally accepted that these roadway discontinuities have some effects on vehicle stability and thus on facility safety, these effects have never been comprehensively quantified. A comprehensive treatment of this problem is presented that was developed by using research conducted previously by the California Department of Transportation and Systems Technology, Inc., and adding a testing program to supplement and extend the earlier work. Recommendations are made for the use of this information to establish maintenance guidelines. Appropriate use of this information by highway engineers can, in time, have a major impact on reducing accidents affected by pavement edges. This can be accomplished by reducing unnecessary maintenance and spending available maintenance funds on areas that have real safety significance.

The repair of deteriorated pavement edges and unpaved shoulders adjacent to pavement edges is a maintenance activity that requires continuous effort by all state and local highway agencies. Although it is generally accepted that pavement edges of excessive height have an effect on vehicle stability and thus on facility safety, this effect has never been comprehensively quantified.

The research described in this paper--the analysis of available literature and the testing program--allows a realistic evaluation of the effect of pavement edges on automotive safety and further defines the effect of certain critical driver and vehicle factors so that the urgency of the need for maintenance can be accurately assessed.

Pavement edges can represent a significant problem in vehicle control. In some cases the problem can be worsened and even made critical by inappropriate forms of caution on the part of drivers (see Figure 1). What may happen in such a critical situation can be described as follows:

1. A vehicle is under control in a traffic lane adjacent to a pavement edge where the unpaved shoulder is lower than the pavement elevation.
2. Because of driver inattention or distraction or for some other reason, the vehicle is allowed to move or is steered into a position in which the right side wheels are just off the paved surface. The right side wheels are now to the right of the pavement edge on a surface elevation below that of the main lane.
3. The driver then carefully tries to steer gently back onto the paved surface without reducing speed significantly.
4. The right front wheel encounters the pavement edge, which prevents it from moving onto the pavement. The driver further increases the steer angle to make the vehicle regain the pavement. The vehicle still does not respond. At this point there is equilibrium between the cornering forces to the left acting on both front tires and the pavement edge force to the right acting as shown in Figure 1a.
5. The critical steer angle is added by the driver, and the right front wheel mounts the paved surface. Suddenly, in less than one wheel revolution, the edge force has disappeared and the right front cornering force may have doubled as a result of increases in the available friction on the pavement and increases in the right front wheel load caused by cornering (Figure 1b).
6. The vehicle yaws radically to the left, pivoting about the right rear tire, until that wheel can be dragged up onto the paved surface. The excessive left-turning yaw continues, too rapid in its development for the driver to prevent penetrating the oncoming traffic lane (Figure 1c).
7. A collision with oncoming vehicles or spin-out and vehicle roll may then occur.

Although this phenomenon does occur on highway facilities, in many cases the same result, vehicle loss of control, may occur without the effect of a pavement edge. A loose or muddy shoulder can have the same effect if the driver overreacts when trying to regain the paved surface. Frequently, a pavement edge of modest height is blamed when excessive steering input is the cause.

LITERATURE SURVEY

The qualitative effect of pavement edges, or so-called lip drop-off, has been understood to some degree for many years. In the Traffic Accident Investigator's Manual (1), published by Northwestern University and originally compiled by Baker, the following statement is found: "Lip drop-off is simply a low shoulder at the edge of a hard pavement. It is important when the shoulder is more than three inches below the pavement...." Based on a telephone conversation with Baker in September 1982, it was determined that this conclusion was reached by informal testing at Northwestern University as early as 1959.

Ivey and Griffin (2) dealt with surface discontinuities including pavement edges in a paper published in 1975. Based on 15,968 accidents in the North Carolina accident file, there was an overrepresentation of the key words associated with a shoulder or pavement edge drop--i.e., dropped, soft, curb, and edge. A Delphi study included in this report ranked pavement edge-shoulder drop-off among the top accident-related pavement disturbances.

A series of tests reported by Nordlin (3) in the mid-1970s included a range of automobile sizes and edge drop conditions from 1.5 to 4.5 in. Nordlin concluded that there was no significant safety hazard for vehicles in mounting edges up to 4.5 in. This work did not include testing of the scrubbing situation, in which the offside tire scuffs along the pavement edge before the steering input causes it to mount the edge. This scrubbing action is the most critical situation. In further testing at the California Department of Transportation (Caltrans), Stoughton (4) observed the effect of a broken asphaltic concrete pavement edge and muddy shoulder on vehicle stability. The tests included small, medium, and large passenger cars and a pickup truck driven at speeds up to 60 mph. Again, pavement edge drops of 1.5, 3.5, and 4.5 in. were tested. Stoughton reached the following primary conclusion: "The pavement drop-offs had little effect on vehicle stability and controllability in all tests." Again, the scrubbing situation was not tested.

Klein et al. (5) produced a report in 1976 that included analyses of accident data, questionnaire surveys, and a variety of both open-loop and closed-loop tests. In closed-loop tests, naive drivers were used. Special efforts were made to achieve the edge scrubbing condition. In tests of edge drops up to 4 in. high, loss of control was encountered at the higher speed levels, generally more than 30 mph.

Another important discovery made by Klein is shown in Figure 2, which illustrates the steering-

Figure 1. Loss of vehicle control caused by driver's attempt to return to the roadway.

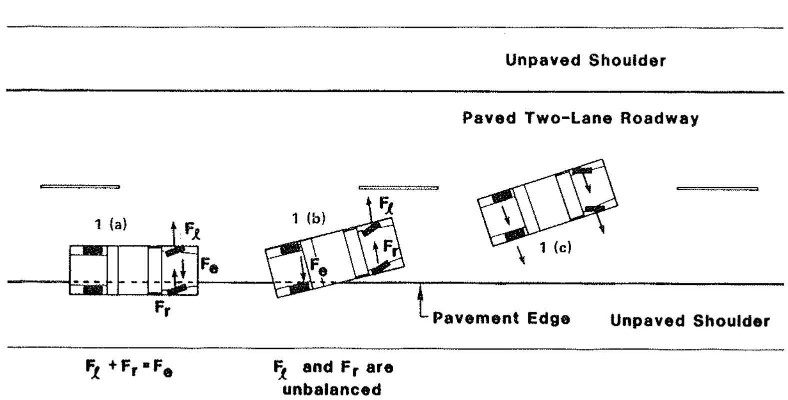
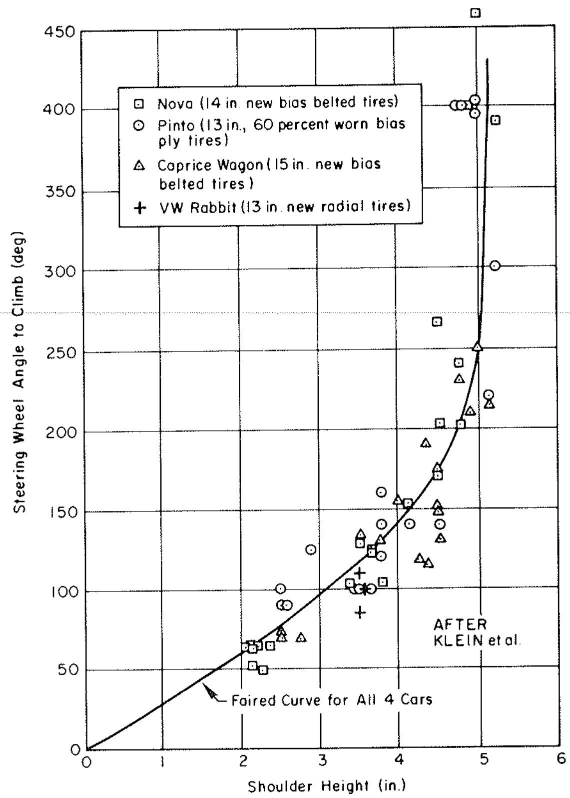


Figure 2. Steering-wheel angle required to climb various edge heights from edge scrubbing condition.



wheel angle required to climb various edge heights from the scrubbing condition (5). The curve in Figure 2 could potentially be used to describe the safety of the maneuver in that the initial relatively linear range (in Figure 2, from 0 to 3 in.) is reasonably safe. As the curve becomes curvilinear, the maneuver becomes significantly more difficult. As the curve starts a precipitous rise, again approaching a straight line, the difficulty of the maneuver becomes extreme.

Klein presented the most analytically appealing and experimentally comprehensive work done on this subject. The main limitation is that he tested only one, albeit the most critical, pavement edge geometric condition--that of an extreme 90 degree angle with little edge rounding. The limitations in scope of the two principal studies--the noninclusion of the edge scrubbing condition in the study by Nordlin and the inclusion of only one pavement edge geometry in the study by Klein--made the present study necessary. The testing plan was designed to complement and extend the earlier studies.

TEST PROGRAM

A comprehensive test program was developed to evaluate the effects of edge conditions based on a variety of drop-off heights, vehicles, tires, drivers, speeds, and positions. The conditions chosen do not represent every conceivable situation involving an edge condition. This would produce an extremely large and unwieldy test matrix. The variables for the initial full-scale testing program were chosen as representative of those typically found on the highways today, which would extend the information already developed by Nordlin (3) and Klein (5).

Edge Height and Shape

To obtain a sufficient number of data points without allowing the test matrix to become too massive, three shoulder-to-pavement heights were chosen--1.5, 3, and 4.5 in.--along with a construction tolerance of ±0.25 in. measured at intervals of 10 ft.

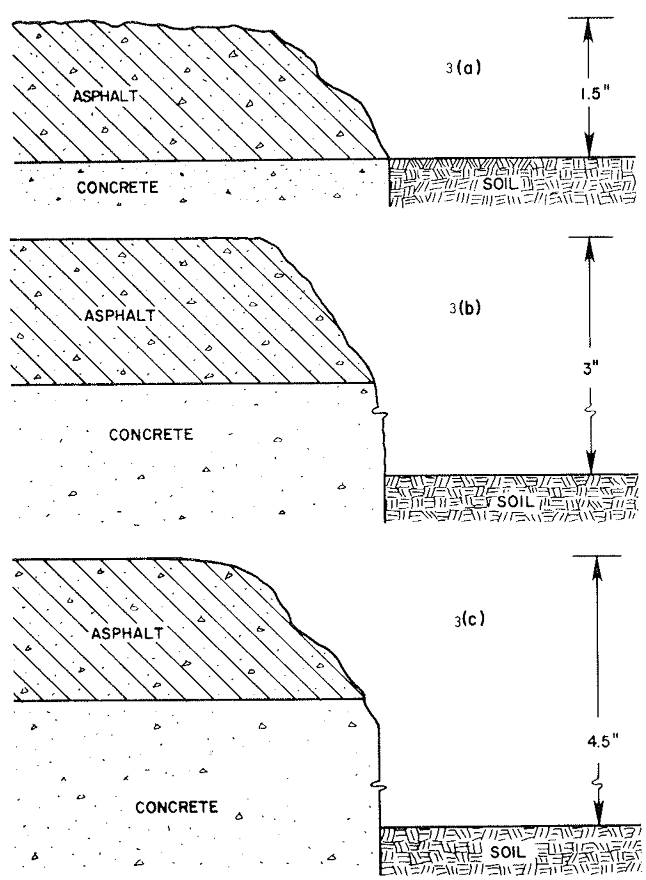
To evaluate these heights, a 500-ft test course was constructed at the Texas Transportation Institute (TTI) Proving Grounds adjacent to an existing concrete runway. To produce a soil shoulder, the vegetation and topsoil were first removed from the runway edge over approximately a 20-ft width. Next, a sandy loam was added, graded for drainage, and finally rolled. With the soil at the level of the existing concrete, a 1.5-in. pad of asphalt, 12 ft wide and 500 ft long, was applied to the concrete. This produced an asphalt-soil interface with a 1.5-in. height.

The edge was left unmodified, as it was installed, to provide a typical edge condition or shape with approximately a 1.5-in. radius. Figure 3a shows the nominal shape of this edge and the slight variations that occur over the length of the test course.

On completion of the 1.5-in. test runs, the soil was cut down 1.5 in. and regraded to produce a 3-in. edge height condition with the same asphalt shape as the 1.5-in. condition and a vertical face 1.5 in. below that, as shown in Figure 3b. The 4.5-in. condition was produced in the same way by removing 1.5 in. of soil and regrading, as shown in Figure 3c.

To gain some insight into edge shape, at the 4.5-in. condition the first half of the course was

Figure 3. Edge profiles.



modified (see Figure 4) to produce a sharper edge with approximately a 0.75-in. radius. By using an epoxy base paving material, with aggregate similar to the asphalt pavement, the required edge was finally produced by grinding to the desired contour and texture. The final edge type tested is the 45 degree slope shown in Figure 5. Informal testing on similar edges had shown the potential of this geometry to reduce the disturbance to a traversing vehicle. The unmodified test course is shown in Figure 6 in the 4.5-in. rounded edge condition and in the 6-in. 45 degree edge condition.

Vehicles

Passenger automobiles and a pickup truck were tested. To evaluate the effect of weight, suspension system, and wheel size, a minicompact, an intermediate-sized automobile, and a full-sized automobile were tested along with a standard-sized pick-up. The vehicles ranged in weight from 1,668 to 4,713 lb and in wheel size from 12 to 15 in. (see Figure 7). Each vehicle was set up to manufacturer's specifications with respect to the suspension and steering system before testing and was periodically inspected during the course of the testing. Each vehicle was equipped with a roll bar and racing lap and shoulder belts to provide an added margin of safety for the test drivers.

Tires

To determine the effect of tire construction, the intermediate- and full-sized sedans were tested with both bias ply and radial tires. The other two vehicles were tested with only radial tires. Only full-tread tires were considered in this testing because smooth or damaged tires could be considered a special case for future investigation. In all cases the tire inflation was adjusted to recommended cold pressures just before testing.

Drivers

Closed-loop tests are those in which the driver is free to steer or brake as needed to maintain stability. Driver skill level is notoriously difficult to assess. To evaluate the effect of skill level, four drivers were chosen:

1. A professional who teaches high-performance driving techniques;
2. A semiprofessional, a technician who occasionally performs as a test vehicle driver;
3. A typical male, a construction supervisor with no special driving skills; and
4. A typical female, a technician with no special driving skills.

Only driver 1 drove the complete matrix of tests, because it was felt that professional skills were required for any tests involving a potentially hazardous situation. Driver 2 performed all tests except those for the 4.5-in. drop-off. Drivers 3 and 4 made a selected number of runs at the 1.5- and 3-in. edge heights.

Test Speeds

To evaluate the effect of vehicle speed, for each test condition runs were made at 35, 45, and 55 mph. These values were chosen to cover the spectrum of speeds that may be encountered in a typical edge recovery situation.

Figure 4. Profile of modified 4.5-in. edge.

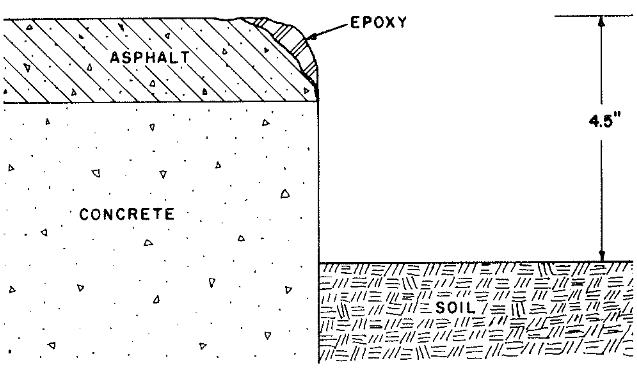


Figure 5. Profile of modified 45 degree edge.

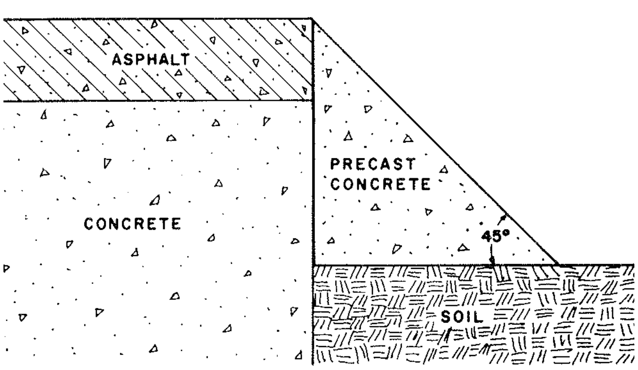


Figure 6. Unmodified test course: (top) minicompact cars on 4.5-in. rounded edge and (bottom) full-sized automobile on 6-in. 45 degree edge.

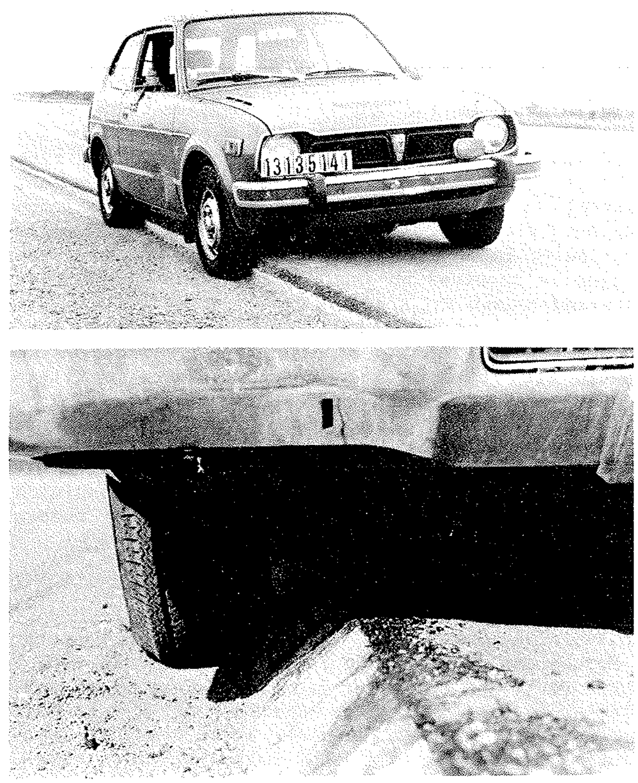


Figure 7. Description of vehicles used in study.

SIZE		MINI COMPACT	INTERMEDIATE	FULL SIZE	PICKUP TRUCK
YEAR		1977	1974	1977	1976
MAKE		HONDA	CHEVROLET	PLYMOUTH	FORD
MODEL		CIVIC	NOVA	GRAND FURY	F150 CUSTOM
Weight(a)	FRONT	L 519 R 514	859 883	L 1347 R 1323	1259 1289
	REAR	L 306 R 329	773 731	1019 1024	872 889
	TOTAL	1668	3246	4713	4309
ENGINE DISPL.		75.5 CID	250 CI	440 CI	390 CID
SHOCK ABSORBERS		TELESCOPING	TELESCOPING	TELESCOPING	TELESCOPING
SUSPENSION		STRUT	BALL JOINTS	BALL JOINTS	KING PINS
POWER STEERING		NO	NO	YES	YES
STEERING RATIO		18.2:1	36:1	21.2:1	21.8:1
BRAKE TYPE/POWER		FT DISC REAR DRUM/NO	DRUM/NO	FT DISC REAR DRUM/YES	FT DISC REAR DRUM/YES
AIR CONDITIONER		NO	NO	YES	YES
TIRE SIZE		P155/80R12	P195/75R14	P225/75R15	L78-15
AND TYPE		GOODYEAR TIEMPO	GOODYEAR POLY STEEL	GOODYEAR VIVA	GOODYEAR POLYGLAS
AVE. TREAD DEPTH		LF 9/32 RF 9/32	LF 11/32 RF 11/32	LF 10/32 RF 10/32	LF 11/32 RF 11/32
		LR 9/32 RR 9/32	LR 11/32 RR 11/32	LR 10/32 RR 10/32	LR 11/32 RR 11/32
RECOMMENDED TIRE PRESSURE		FRONT 24	FRONT 24	FRONT 30	FRONT 30
		REAR 24	REAR 24	REAR 30	REAR 36
WHEELBASE		86.5"	111.25	122	132.75
FRONT TRACK		51.5"	59.25	63.875"	65
REAR TRACK		50.75"	59.5	63.625"	64.5
MILEAGE		70122	07077	78651	65270
MINIMUM GROUND CLEARANCE mm		5.25"	6.25"	7.25"	8.75"

(a) Weight Less Driver and Instruments

Vehicle Positions

Once a vehicle has left the roadway and has a wheel or wheels on the shoulder, several methods of returning to the roadway are possible. Three of these methods were investigated during the study.

The first is the scrubbing condition, in which the driver allows the vehicle to move laterally toward the roadway at a very slow rate until a tire contacts the roadway-shoulder lip. At that point lateral motion stops and the tire is in intimate contact with the roadway edge (scrubbing) while continuing to travel forward. To mount the edge the driver is required to input an increasing amount of steering toward the roadway.

In the second condition, the right front and rear wheels are well on the shoulder while the other two wheels are still on the roadway. The driver then steers to the left at a comfortable level to produce a lateral velocity high enough to preclude any continuous scrubbing condition as the vehicle returns to the roadway. In the third condition, the vehicle returns to the roadway as in the second condition except that all wheels are on the shoulder before the return maneuver is initiated.

Subjective Rating System

In addition to the photographic and electronic data, a system was developed to allow the driver to report the severity of each test run immediately after completing it. This system consisted of a numerical ranking from 1 (no detectable effect) to 10 (complete loss of control). The following list, referred to as a severity code, was prepared to assist the drivers in assigning a number to their impressions:

1. Undetectable;
2. Very mild;
3. Mild;
4. Definite jerk;
5. Effort required;
6. Extra effort;
7. Tire slip (slight lateral skidding);
8. Crossed centerline and returned;
9. Crossed centerline, no return; and
10. Loss of control (spin-out).

Even though this system is subjective and tends to vary from driver to driver, it proved a good indicator when confined to any one driver's reactions to the entire matrix of tests. This rating value was later used as the dependent variable in sorting by computer on various combinations of conditions.

DISCUSSION OF RESULTS

To evaluate the effect of the three edge heights on two modes of returning to the roadway (scrubbing and a smooth return), average severities were obtained by using the professional driver. Only this driver was used for these data because he completed the entire test matrix.

Figure 8 shows the average values for each of the conditions when the vehicle smoothly returns to the roadway from a position in which it is about half on the earth shoulder and half on the pavement. Clearly, there is little difference between either vehicles or edge heights (3 versus 4.5 in.).

Figure 9 shows the same series of tests for a different prereturn condition. In this case the right wheels are in intimate contact with the pavement edge (the scrubbing condition) before the return to the pavement is attempted. Once again the

Figure 8. Comparison of vehicle performance in nonscrubbing condition at 35, 45, and 55 mph.

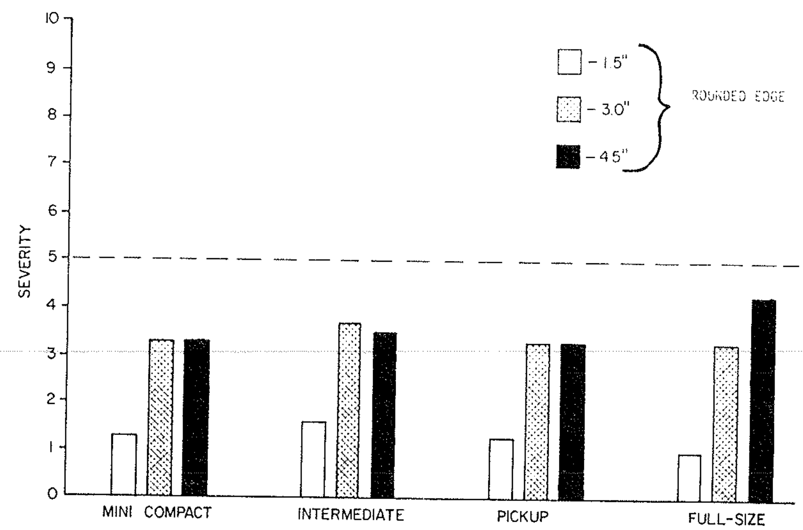
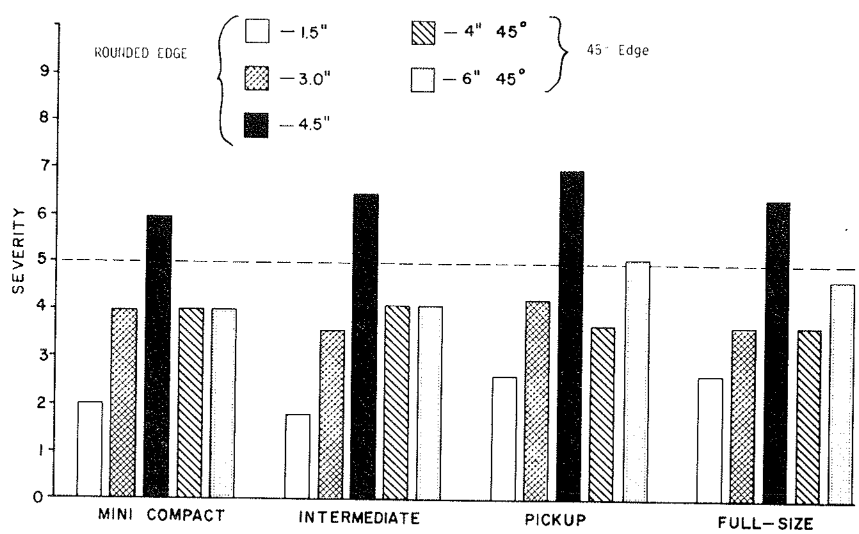


Figure 9. Comparison of vehicle performance in scrubbing condition at 35, 45, and 55 mph.



difference between the vehicles is quite small. But the effect of edge height on the rounded edge is pronounced in the case of the 4.5-in. height: Figure 9 shows the severity of the return maneuver for all vehicles at this edge height extending into the upper half of the graph--i.e., the critical severity range. There is a marked reduction in severity for the 4- and 6-in., 45 degree edges compared with the rounded edges.

To determine the effect of driver skill level, average severity levels were obtained from three drivers (the professional, the semiprofessional, and the untrained male). Data from all vehicles at each test speed for the 3-in. edge height were summarized. The average values for each driver under each prereturn condition were fundamentally equal.

Because both bias and radial tires were tested on the intermediate- and full-sized vehicles, it was possible to make a comparison by using the similar conditions of each test speed, the professional driver, and the intermediate-sized vehicle at the various edge heights. Only the scrubbing condition was considered in this comparison because it involves tire construction much more than the other two conditions. The modified 4.5-in. edge (0.75-in. radius) was also used in this comparison. The bias ply was found to produce slightly higher severity levels than the radial at all edge heights.

Finally, the effect of the speed at which the vehicle returned to the roadway was considered (see Figure 10). Only the runs made by the professional driver were evaluated to maximize the scope of the comparison. In addition, only the scrubbing condition was considered because it has been shown to lead to the most hazardous conditions.

The test results for all vehicles were averaged. As Figure 10 shows, within each edge-height condition a nearly linear increase in severity occurs as the speed increases. As before, the 4.5-in. edge height is a potentially unsafe condition, even at the 35-mph speed. It should also be noted that 45 and 55 mph exceed the critical speeds found by Klein (5). In the case of a 6-in., 45 degree edge, a condition approaching a severity rating value of 5 occurs as the vehicle speed approaches 55 mph.

Steering Angle

Steering-wheel movements required to perform a specific maneuver is one of the most graphic means of developing an appreciation of the difference between a relatively safe and a potentially hazardous condition. The difference between the maneuvers for 3-in. and 4.5-in. edge heights is shown in Figures 11 and 12. Figure 11 shows the results of a typical run made by the professional driver at 55 mph in the intermediate-sized vehicle. In this run, the driver returns to the pavement over a 3-in. drop height. Figure 12 shows the results of a run made under the same conditions but at the 4.5-in. edge height. Notice the extreme arm movements of the driver at 4.67 and 5.75 sec in Figure 12.

Figure 10. Averaged test results for all vehicles in scrubbing condition at 35, 45, and 55 mph.

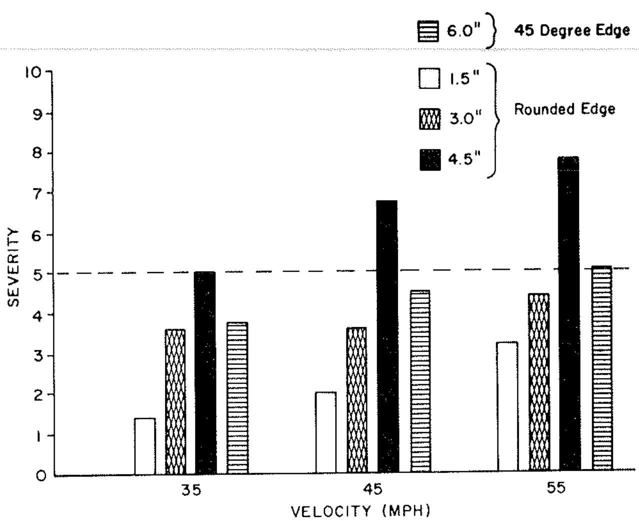
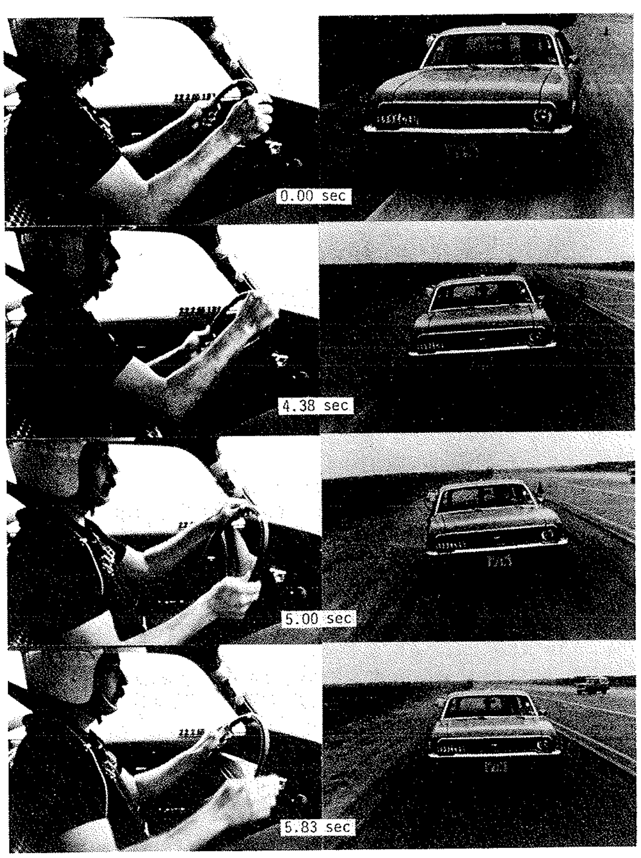


Figure 11. Steering-wheel maneuvers by professional driver in scrubbing condition: 3-in. edge height, intermediate-sized vehicle, at 55 mph.



The amount of lateral acceleration generated after returning to the roadway to avoid entering the oncoming lane of traffic has a significant effect on safety. Figure 13 shows the average lateral acceleration developed over the three speed ranges by the four test vehicles at the various edge heights. The two conditions of prerecovery position are considered. Driver opinion levels obtained from Kummer and Meyer (6) are indicated on the chart. A level of lateral acceleration greater than 0.3 g is considered excessive. This would be a potential limitation imposed on themselves by most drivers, who may not attempt to develop all of the available friction to remain in their lane of traffic. The scrubbing condition produced quite high values--0.7 to 0.8 g--for the 4.5-in. edge, which proved to be near the maximum available friction levels because lane violations and slipouts did occur.

The results of the non-scrubbing-condition test runs compare favorably with the edge mounting work done by Nordlin (3). No loss of control of the test vehicle and no lane encroachment up to and including the 4.5-in. edge height occurred in Nordlin's study or in the research reported here.

The influence of the edge shape is brought out by comparing the results of this study with the results of a study by Klein (5), who used a smooth, near vertical concrete surface for an edge with a 0.5-in. radius at the top. This edge produced similar steering angles for the return maneuver from a scrubbing condition at the 4.5-in. height. Klein reported a steering angle of 9 degrees compared with the 7 degrees observed in this study. The large difference shows up at the lower heights, where 2.5 and 5 degrees were required on the sharp edge at the 1.5- and 3-in. heights compared with the 1.5 degrees needed on the more rounded asphalt edge used in this project.

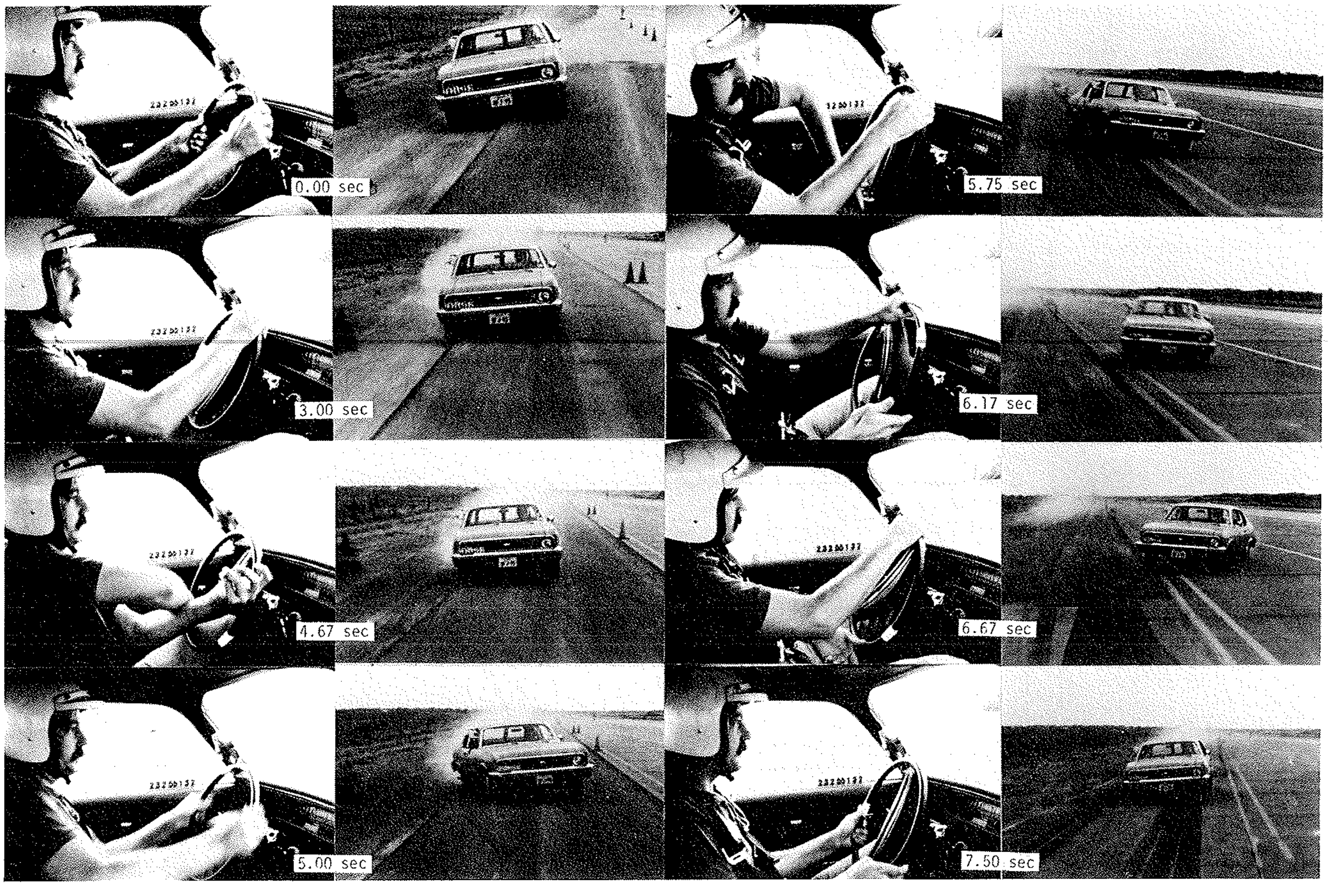
LIMITATIONS OF THE TEST PROGRAM

Although the choice of vehicles would seem to be adequate to define a fairly wide spectrum of vehicle-handling characteristics, this has not been experimentally verified. To evaluate objectively the spectrum of vehicles covered, parameters such as inertial properties, spring stiffness, and tire cornering stiffness should be defined. Other factors, such as vehicle loading and the effect on vehicles other than four-wheeled automobiles, were not considered. The effect of differential tire pressures, especially low pressures on one or more rear tires, was not specifically investigated, although it is anticipated that anything that enhances oversteering would be more critical in a scrubbing edge maneuver. It should also be noted that the subjective rating technique was attempted with only four subjects. This is a limitation, although it was surprising how consistent the ratings were within that group. Although it has not been possible to prove, by using vehicle accident statistics, whether the scrubbing situation is in fact a contributor to a significant number of accidents, in an effort to be conservative that test condition is thought to be a reasonable indicator for use in the development of maintenance guidelines.

SUMMARY AND CONCLUSIONS

The results of the study are summarized in Figure 14, where relative degree of safety, in terms of the subjective severity levels defined previously, is plotted versus change in longitudinal edge elevation. Three curves are shown, one for each pavement edge profile shape: Shape A is the sharp edge tested by Klein, shape B is the rounded edge shown in Fig-

Figure 12. Steering-wheel maneuvers by professional driver in scrubbing condition: 4.5-in. edge height, intermediate-sized vehicle, at 55 mph.



ure 3, and shape C is the 45 degree edge shown in Figure 4b.

The terms used to describe relative degrees of safety can be defined as follows:

1. Safe indicates that, no matter how impaired the driver or how defective the vehicle, the pavement edge will have nothing to do with a loss of control. (This includes the influence of alcohol and other drugs and any other infirmity or lack of physical capability.) The term includes the subjective severity levels 1 through 3.
2. Reasonably safe indicates that a prudent driver of a reasonably maintained vehicle would experience no significant problem in traversing the pavement edge. The term includes subjective severity levels 3 through 5.
3. Marginally safe indicates that a high percentage of drivers could traverse the pavement edge without significant difficulty. A small group of drivers might experience some difficulty in performing the scrubbing maneuver and remaining within the adjacent traffic lane. The term includes subjective severity levels 5 through 7.
4. Questionable safety indicates that a high percentage of drivers would experience significant difficulty in performing the scrubbing maneuver and remaining within the adjacent traffic lane and full loss of control could occur under some circumstances. The term includes subjective severity levels 7 through 9.
5. Unsafe indicates that almost all drivers would experience great difficulty in returning from the pavement edge scrubbing condition and loss of control would be likely. The term includes subjective severity levels 9 through 10.

Figure 14 represents an effort to summarize all research and testing results available for interpretation, including the testing by Caltrans, Systems Technology, Inc., and TTI. It was necessary to use subjective judgment in constructing this figure, based on the preceding definitions of safety. The figure is subject to the limitations previously stated and represents average vehicle and speed conditions. Atypical vehicles, vehicle loadings, tire conditions, and speeds could result in significant shifts in the positions of the curves.

Figure 14 could have direct application to maintenance recommendations. For example, consider the point on the shape B curve where the curve crosses line 1--i.e., at the 2.5-in. elevation change. This height might indicate a need to prepare for maintenance before the edge level increases to the point where the curve crosses line 2--at 3.5 in. For shape A edges, maintenance would be somewhat more critical: maintenance activities are indicated at a height between 2.0 and 3.0 in., roughly corresponding to the crossing of lines 1 and 2.

The advantage of avoiding shape A is also apparent in Figure 14. If shape C can be constructed, either during the original construction or as a maintenance activity, the need for edge maintenance could be significantly reduced. It is thought that these curves can be used as a guide for the maintenance of pavement edges and that they indicate the desirability (based on determinations of cost-effectiveness) of gradually moving toward edge shape C. Shape C may also result in significant gains in the reduction of edge deterioration.

Using data from Michigan's Highway Safety Research Institute, Indiana data, and his own survey questionnaire, Klein (5) has shown pavement edges to

Figure 13. Maximum lateral acceleration and edge height.

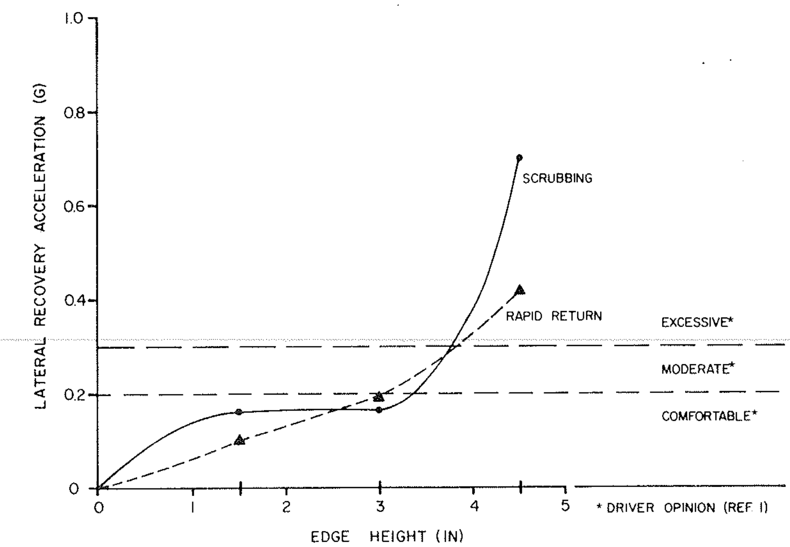
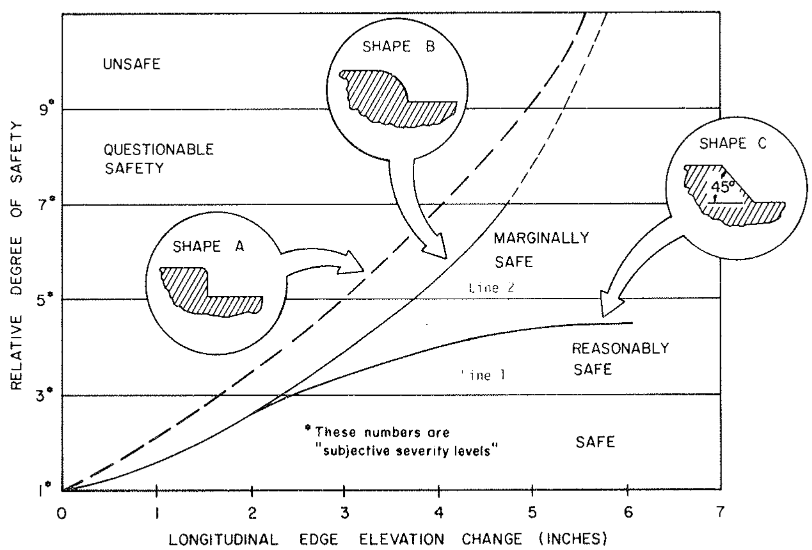


Figure 14. Relationship between edge geometry and safety for scrubbing condition.



be the most significant safety-related roadway disturbance. Klein's research was the most significant after the California data and was among the top two in importance in earlier studies by TTI. Appropriate use by highway engineers of the information contained in these studies can, in time, have a major impact on reducing accidents affected by pavement edges. By establishing maintenance guidelines based on the findings shown in Figure 14, unnecessary maintenance of shoulders can be reduced and available maintenance funds can be concentrated in areas that have real safety significance.

ACKNOWLEDGMENT

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REFERENCES

1. J.S. Baker. Traffic Accident Investigator's Manual, 4th rev. ed. Traffic Institute, Northwestern Univ., Evanston, Ill., 1975.
2. D.L. Ivey and L.I. Griffin III. Driver Vehicle Reaction to Road Surface Discontinuities and Failures. Presented at International FISITA Conference, Tokyo, 1975.
3. E.F. Nordlin et al. The Effect of Longitudinal Edge of Paved Surface Drop-Off on Vehicle Stability. California Department of Transportation, Sacramento, Rept. CA-DOT-TL-6783-1-76-22, March 1976.
4. R.L. Stoughton et al. The Effect of a Broken A.C. Pavement Drop-Off Edge and Muddy Shoulder on Vehicle Stability and Controllability. California Department of Transportation, Sacramento, Memorandum Rept., July 1978.
5. R.H. Klein, W. A. Johnson, and H. T. Szostak. Influence of Roadway Disturbances on Vehicle Handling. U.S. Department of Transportation, Oct. 1976.
6. H.W. Kummer and W.E. Meyer. Tentative Skid-Resistance Requirements for Main Rural Highways. NCHRP, Rept. 37, 1967.