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Contents

A New Method of Traffic Evaluation For Pavement Design

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Most procedures currently used for structural and geometric design of pavements fail to treat adequately effects of mixed traffic on pavement life. The effects of magnitude, configuration, and number of repetitions of various wheel loads should be included as variables in any good pavement design procedure.

An analysis of axle loads observed on Texas highways is used with equivalence factors developed at the recently completed AASHO Road Test to develop a practical method for analyzing the damaging effects of mixed traffic. The method of traffic evaluation can be adapted for use with many different design procedures.

• WITHIN the last twenty years, the methods used to determine the design thickness of each layer in flexible pavements have changed from procedures based almost solely on the experience and judgment of one individual to procedures which utilize many scientific techniques.

All of the design methods take into consideration in some form the relative strength properties of the roadway materials, the effect of traffic, and the effect of time. It is difficult, however, to differentiate and explain clearly how some procedures take the variables of material properties, traffic, and time into consideration.

On reviewing many design procedures it is apparent that most methods of evaluating traffic are completely empirical. Most methods fail to account for three important elements: (a) the effect of the number of repetitions of wheel loads, (b) the effect of variations in the magnitude of wheel loads on in-service roads, and (c) the effect of volume, or the number of each specific wheel load by weight.

The apparent attitude of resignation among pavement designers concerning the evaluation of the effects of mixed traffic has not occurred without reason. There are three predominant extenuating circumstances which contributed to this.

First, procedures developed prior to about 1940 were based on design considerations for roads carrying relatively small volumes of traffic with few heavily loaded vehicles. The use of a static loading test to evaluate the behavior of a soil which would be subjected to dynamic loads on the road proved to be satisfactory in the early years, and design procedures such as the CBR method resulted in pavements which stood up under traffic for a period of time. There appeared to be no reason to worry about precise analysis of the effects of traffic.

Second, and probably the most fundamental factor, there was no scientific method of defining the serviceability of pavements. As a result of this deficiency, the damaging effect of a specific wheel load could not be evaluated accurately. An analysis of mixed traffic on in-service roads further complicated the problem.

Third, there was no method for evaluating the effect of the variables other than those directly associated with traffic. The effect of construction control and variations of

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both temperature and moisture in the area were but a few of the unresolved conditions which further complicated the problem and contributed to an attitude of resignation.

These extenuating circumstances have for the most part been overcome. The AASHO Road Test has been completed and the results are being studied by highway engineers everywhere. The "AASHO Interim Guide for the Design of Flexible Pavement Structures" and the "AASHO Interim Guide for the Design of Rigid Pavement Structures" are summaries of the AASHO Road Test data prepared by the AASHO Committee on Design. These Guides provided a tool by which pavement designers may combine local experience with experience gained at the AASHO Road Test.

This paper discusses the development of a practical traffic analysis which may be used to analyze mixed traffic for pavement design.

AASHO Road Test

The most thorough evaluation of the effect of traffic on pavement performance to date is contained in the reports of the AASHO Road Test. The equivalence factors developed represent an extensive evaluation of the relative destructive effect of traffic. The definition of an equivalence factor is "the number of applications of an 18,000-lb single-axle load required to produce the same effect on the serviceability of a pavement as one application of a particular axle load."

W. N. Carey, Jr., Chief Engineer of Research on the AASHO Road Test, made the following comment in respect to the equivalence factors:

> The most important result of the physical tests at Ottawa was the one by which the relative effects of different axle loadings on pavement performance can be determined. This could not have been found by study of ordinary highways in operation. Controlled traffic, as at the Road Test, where only one axle load operated over any given test pavement, was essential. This controlled traffic made it possible to assign responsibility for damage to a specific load. The effect of specific loads on performance was so clearly shown that it is possible to evolve very rational theories as to the probable effects of mixed traffic on pavement performance. . . The Road Test could not show whether or not the load effect would be the same in other environments as it was at Ottawa. On the other hand, without another controlled traffic test it cannot be shown that they would not be. I suggest that pavement designers might accept the load effect relationships resulting from the Road Test as good approximations of the load relationships everywhere until such time as another controlled traffic research is undertaken. If this advice is accepted, highway engineers can immediately look to the entire highway system as testing ground for study of the many design relationships that are still unknown. Mixed traffic, if the Road Test equivalencies are used, will be a perfectly satisfactory treatment for numerous experiments involving pavement design parameters. It is only required that the axle loads and frequencies of the mixed traffic be determined with fair precision.

The succeeding sections present a method of determining the axle loads and their frequencies in mixed traffic.

Definitions and Abbreviations

Tandem-Axle Set — Two single axles whose centers are included between two parallel transverse vertical planes 40 in. apart, extending across the full width of the vehicle.

Axle Load — The total load transmitted to the pavement structure by all wheels of either a single axle or a tandem-axle set.

Design Interval — That period of time required for the design traffic to develop.

Serviceability — The ability, at the time of observation of a pavement to serve high-speed high-volume automobile and truck traffic.

EVALUATION OF TRAFFIC

Traffic Projection

The first step in the evaluation of traffic for use in highway design requires an analysis of the facility proposed. Past traffic history, land use, relationship to the area's long-range transportation plan, and other factors must all be considered. Once these factors are evaluated and weighed, a traffic projection for the design interval can be made.

Figure 1 depicts a hypothetical traffic projection wherein a facility with a 20-yr service life is being analyzed for improvement. The average daily traffic has been projected 25 years and the design interval has been selected as 20 years. It should be noted that Figure 1 uses the word "axles" synonymously with ADT. This does not mean to imply that these two are directly interchangeable, but rather shows that axle counts can be projected directly because the "raw" traffic data are normally obtained in axle counts. The projection of axles rather than ADT at this point simplifies the overall mechanics of this program; however, in order to stay with current terminology, ADT is used.

Attention is called to the needed additional study of the reliability of traffic sampling and projections. Statistical analysis in the number and location of sampling stations and in the size and time of sample should be conducted on a continuing basis as should a comparison of earlier predictions with current traffic.

Relationship of Percent Trucks to ADT

The second step in the evaluation of traffic was to establish the characteristics of mixed traffic found on in-service roads in Texas and several states. A study of all the manual classifications and count stations in Texas for the period 1951-1961 was made and the percent truck traffic was determined. Pickups and panel trucks which have relatively the same performance and weight characteristics as automobiles were not included as part of the truck traffic.

Figure 2 shows the result of this study. The "upper limit" line represents approximately the highest percent of trucks that has been experienced for any given ADT on

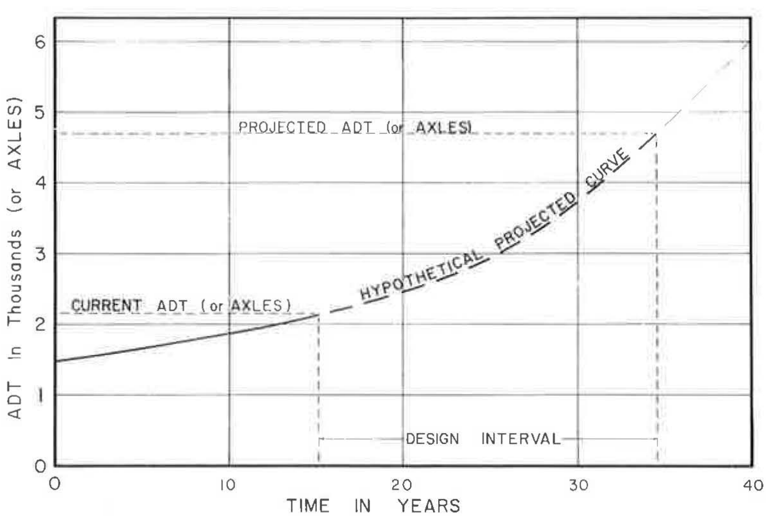

Figure 1. Traffic projection curve.

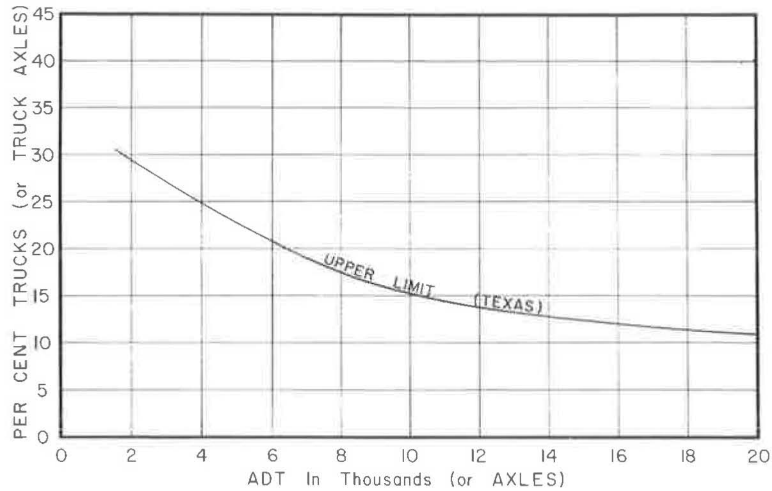

Figure 2. Relationship of percent trucks to ADT.

Texas highways during the 1959-61 period. Approximately 96 percent of all the stations plotted were below the line. The majority of the stations which were above the line had less than 4,000 ADT. Very few stations had less than four percent truck traffic.

Whenever traffic analysis and projections are made, a curve depicting the change in percent trucks to ADT should be established for the section of highway being considered. In Texas this curve would fall between a flat four percent truck traffic line and the upper limit line and normally would have the characteristics of the upper limit line.

Figure 2 also uses the work "axles" synonymously with ADT. Again, the term "axles" could be used directly in lieu of the more familiar term of ADT and thereby simplify the overall mechanics of this program.

Attention is called to the need for additional study of the reliability of the data and for a sampling program to detect local trends.

Single Axles and Tandem-Axle Sets per Hundred Vehicles

The third step (Table 1) shows the current conversion factors used on the primary highway system for changing ADT to axles. This sample tabulation has been developed from current traffic studies. It should be continually expanded and revised once this program development is completed. This step could be included later in the program if the term "axles" had been used in lieu of ADT in Figures 1 and 2. The third step would then be required to convert axles to the projected ADT and the projected percent trucks necessary in geometric design considerations.

Distribution of Axle Loads

An investigation of the wheel load data from the 21 loadometer stations operated by the Planning Survey Division of the Texas Highway Department resulted in a family of curves which describes the characteristics and distribution of axle loads of mixed traffic found on in-service roads in Texas. Figures 3 and 4 were developed from the data of 21 loadometer stations operating across the state in 1960. It is necessary that both figures be used with each other in order to have a complete picture of the distribution. There are several points to be noted on these two figures.

Single-Axle Distribution (Fig. 3).—As would be expected, a highway with a low percentage of truck traffic normally has high traffic volumes. More than 80 percent of all the axles on this road are single axles weighing less than 2 kips. Approximately 0.06

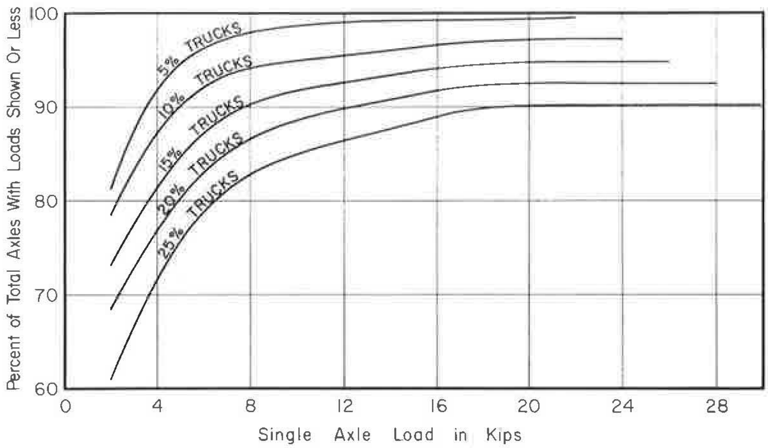

Figure 3. Single-axle distribution at 21 loadometer stations in 1960.

percent of all the single axles exceed the 18-kip legal limit, but usually do not exceed 22 kips.

A highway with a high percentage of truck traffic normally has low traffic volumes. Less than 60 percent of all the axles are single axles weighing less than 2 kips. There is a considerable increase (approximately 0.15 percent) in the number of single axles that exceed the 18-kip legal limit, with the maximum axle weight usually not exceeding 30 kips.

Tandem-Axle Distribution (Fig. 4).—A highway with a low percentage of truck traffic has a very low percentage (approximately 0.74 percent) of the total axles of tandem-axle sets. The tandem-axle sets that are overweight, approximately 0.05 percent, usually do not exceed 42 kips. Conversely, a highway with a high percentage of truck traffic normally has low traffic volumes. Approximately 10.10 percent of the total axles are tandem-axle sets. Those that are overweight, approximately 0.16 percent, usually do not exceed 50 kips.

Attention is called to the need for additional study on a recurring basis of: (a) the reliability of loadometer sampling, (b) the statistical analysis of the number and location of sampling stations, and (c) the size and time of sample.

In the past, high traffic volumes have generally been associated with heavy wheel loads. This relationship appears to be changing. Following the 1959 legislative raising of the maximum legal gross load limit to 72,000 lb per truck, there has been an appreciable increase in the number of tandem-axle sets and accordingly a decrease in single axles.

TABLE 1

NUMBER OF SINGLE AXLES AND TANDEM-AXLE SETS FOR EACH TYPE OF VEHICLE PER EACH 100 VEHICLES FOR THE PRIMARY HIGHWAY SYSTEM

Vehicle Classification	0 - 3,999 ADT, 20 % Trucks		4,000 - 7,999 ADT, 18 % Trucks		8,000 - 11,999 ADT, 14 % Trucks		12,000 - 15,999 ADT, 12 % Trucks		16,000 - 19,999 ADT, 10 % Trucks		20,000 - 24,999 ADT, 8 % Trucks		25,000 & Over ADT, 6 % Trucks	
	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets	Single Axles	Tandem-Axle Sets
Single unit vehicles:														
Passenger cars	143.60	-	147.20	-	154.40	-	158.00	-	161.60	-	165.00	-	168.60	-
Panel & pickup trucks	16.40	-	16.80	-	17.60	-	18.00	-	18.40	-	19.00	-	19.40	-
Other 2-axle vehicles	11.60	-	10.40	-	8.20	-	7.00	-	5.80	-	4.60	-	3.60	-
3-axle	0.40	0.40	0.40	0.40	0.30	0.30	0.30	0.30	0.20	0.20	0.20	0.20	0.10	0.10
Combinations:														
3-axle	6.30	-	5.70	-	4.50	-	3.90	-	3.00	-	2.40	-	1.80	-
4-axle	10.80	5.40	9.60	4.80	7.60	3.80	6.40	3.20	5.40	2.70	4.40	2.20	3.20	1.60
5-axle	6.30	12.60	5.70	11.40	4.30	8.60	3.70	7.40	3.20	6.40	2.50	5.00	1.90	3.80

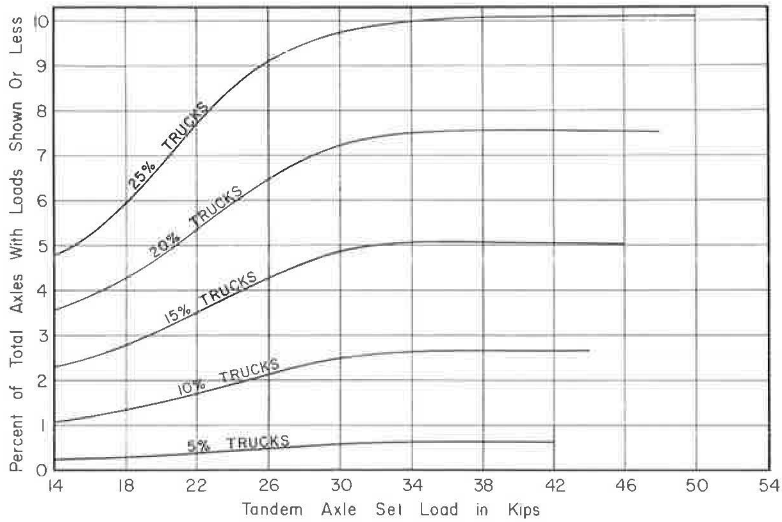

Figure 4. Tandem-axle set distribution at 21 loadometer stations in 1960.

The addition of tandem-axle sets was the only practical way in which the gross load could increase and still conform to the unchanged requirements of a maximum load of 18,000 lb per single axle and 32,000 lb per tandem-axle set.

These relationships in axle load weight and the arrangement of axles, must be continually reviewed and revised for these curves to be representative and for changes in them to be recognized.

Program Block Diagram

A cursory review of the degree of accuracy with which the parameters involved in designing pavement structures are presently defined reveals that it is unnecessary, and impractical as well, to determine the effect of every specific axle load to which a road will be subjected during its design life. Likewise, to determine the exact number of axles of each magnitude and to multiply each specific axle by its respective equivalent factor would be unnecessary in view of the accuracy desired.

Pavement design will not require such a refinement in traffic evaluation without further improvement in other phases pertinent to highway construction. However, caution must be exercised to insure that designers do not continue to minimize traffic analysis now that the relative destructive effect of wheel loads can be evaluated.

The development of a computer program by which traffic may be analyzed and converted to an equivalent number of 18,000-lb single-axle load applications by simple calculations is now in order, utilizing the previously discussed relationships. Figure 5 schematically ties these relationships together.

Input.—Input will normally consist of the initial year's ADT and the design interval along with established boundary conditions for traffic projections (Fig. 1) and percent trucks (Fig. 2). Relationships established in these first two figures will have been based on the analysis of past experience and at this point it will be a matter of selecting the desired boundary.

Traffic Projection Curve (Fig. 1).—ADT is projected by one or more selected curves for the selected design (time) interval.

Relationship of Percent Trucks to ADT (Fig. 2).—The percent trucks is projected by one or more selected curves not exceeding the "upper limit" line. Entering Figure 2 with the ADT projected from Figure 1, the corresponding percent trucks on the ordinate axis can be found.

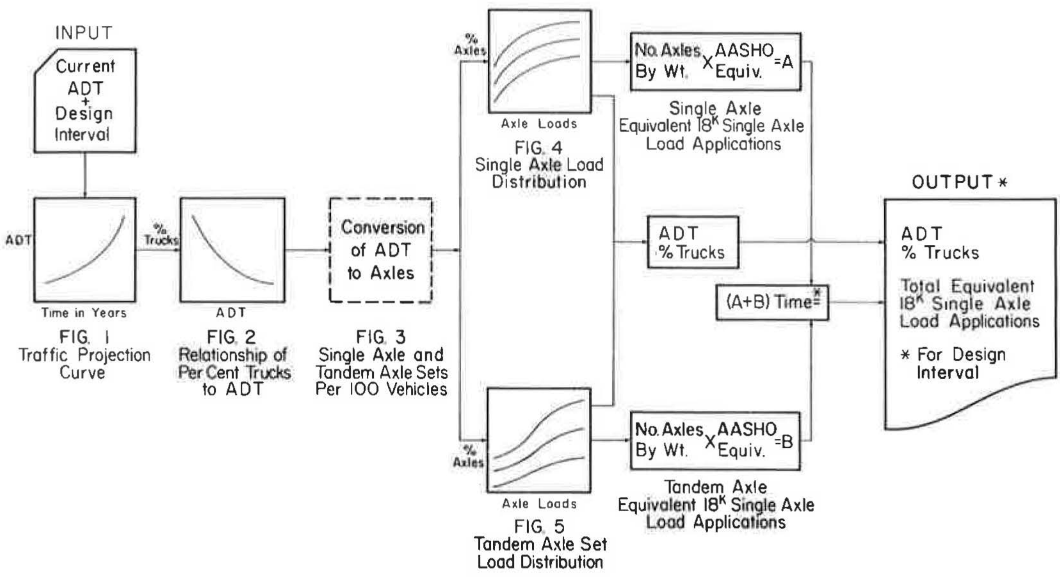

Figure 5. Program block diagram.

Single-Axle and Tandem-Axle Sets per 100 Vehicles (Table 1).—ADT is converted to axles based on the following parameters: ADT volume group, percent trucks, and highway classification.

Distribution of Axle Loads (Figs. 3 and 4).—The total number of axles are distributed according to type and weight with single axles in 2-kip increments and tandem-axle sets in 4-kip increments.

Equivalent 18-Kip Single-Axle Load Application.—The total number of single axles in each weight group is multiplied by its respective AASHO Road Test equivalence factor. Similarly, the total number of tandem-axle sets in each weight group is multiplied by its respective AASHO Road Test equivalence factor. All these equivalent 18-kip single-axle load applications are added for the total design interval to provide the "total equivalent 18-kip single-axle load applications" for which to design the pavement structure.

The ADT and percent trucks necessary for geometric design considerations are also provided for the design interval selected.

The traffic analysis developed in this study provides a practical method of evaluating the effects of all wheel loads on roads subjected to mixed traffic. The analysis for mixed traffic developed herein can be adapted for use with any currently accepted design procedure.

DISCUSSION AND CONCLUSIONS

The primary purpose of this study was to develop a practical method of evaluating the effects of all wheel loads on roads which carry mixed traffic. Equivalence factors derived from experience at the AASHO Road Test were used to convert wheel loads of the type observed on Texas roads to an equivalent number of 18,000-lb single-axle loads.

It may be pointed out that the analysis of traffic reported herein is based on data collected at 21 loadometer stations in Texas during 1958 and 1960. As more experience in analyzing mixed traffic is gained, modifications to the limits selected for axle weight groups and to the magnitude of the equivalence factors used might be required; however, the techniques employed in the traffic analysis as developed in this report can be used to make these modifications.

The wheel load analysis is suitable for evaluating all traffic on present intercity highways in Texas. The method of analysis is adaptable to all types of roads, irrespective of volume or wheel load size. Traffic on urban expressways with large traffic volumes and relatively few heavily loaded trucks, primary highways with medium traffic volumes and heavy axle loads may be considered. The analysis could also be utilized for county roads with extremely low traffic volumes and an occasional heavy load, or a city street with high traffic volumes and almost no heavy loads.

The analysis provides a research tool by which old traffic records can be used to analyze past pavement performance. The backlog of experience and data from old roads should be used to further check and improve this analysis.

The new analysis of traffic when compared with current design procedures results in the following general trends:

1. Design life will now be considered as that time period required for design traffic to develop. In designing the pavement structure, engineers can design for a selected number of repetitions of wheel loads rather than for a selected period of time.
2. Stage construction can now be a programed item. A structure may be designed to full standards; however, the construction of part of the surface course may be delayed until traffic volumes merit additional surfacing.
3. A realistic pavement rating system can perhaps now become a reality. A serviceability index defined at the AASHO Road Test and the AASHO Road Test equation which rated the performance (the trend of serviceability with load applications) of pavements on the Road Test may now be applied to satellite road tests and studies.

RECOMMENDATIONS

In concluding this study the author recommends that the following studies should be conducted to refine the traffic analysis developed in this report.

1. Loadometer sampling techniques should be re-evaluated in conjunction with this analysis because extensive coverage and statistically sound data will be necessary to determine adequately axle load distribution for use in the analysis.
2. Future traffic studies should investigate the character of axle loads on the various lanes of multi-lane facilities to determine properly the total number of equivalent 18-kip single-axle load applications for which each lane of the roadway should be designed. Although it is a relatively safe assumption that the heavier units travel over the slower lanes, current speed studies indicate that heavy trucks are capable of maintaining average speeds of 50 mph. The normally faster, interior lanes are now experiencing heavier wheel loads.
3. A computer program as recommended to analyze past and future traffic data should include the development of equations from which projects of ADT, percent trucks and axle load distribution could be made.

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Appendix

GATHERING OF TRAFFIC DATA IN TEXAS

The Planning Survey Division, usually referred to as File D-10, collects all traffic data for the Texas Highway Department. The Division's data-gathering operation can be divided into three sections.

1. Twenty-one permanent loadometer (vehicle wheel weighing) stations located in the eastern two-thirds of the state. Although the station locations inadvertently give the appearance of being geographic, the criteria for selecting station sites are based on intercity traffic volumes. Each station is operated for one 8-hr period each month, 4 hr for incoming traffic and 4 hr for outgoing traffic. The sampling periods are scheduled from 6 AM to 2 PM, 2 PM to 10 PM, and 10 PM to 6 AM. Every three months a complete 24-hr cycle is completed. The stations are operated 36 hr in each direction for a total of 72 hr per year. During each 4-hr session all trucks and approximately 10 percent of all pickups are weighed. Cars are not weighed. The loadometer crew is composed of six men: a party chief supervises the operations; a flagman directs traffic approaching the station; two men weigh the outside half of each axle with a portable pit scale, measure the height of the vehicle and interview the driver as to the commodity carried and the origin and destination of the trip; and one man makes a visual, two-directional traffic count and classifies the vehicles by type, number of axles and axle configuration.

2. One hundred and forty automatic continuous count stations are located throughout the state. Recording counters are used at each station to count and record all axles, 24 hr a day all year long. The count is photographed every hour on film strips. These film strips are mailed to Austin every week.

3. One hundred and sixty-eight permanent manual traffic count and classification stations are located throughout the state. A visual traffic count and vehicle classification by type, number of axles and axle configuration are made at each station. Forty-eight of these stations are operated six times a year for 24-hr periods. The permanent manual count and classification stations are often supplemented by periodic spot counts of all roads on the state system. Reports and projections based on these data are prepared by the Planning Survey Division for use by the Department.

An Interpretation of Vibration Tests on Roads By the Impedance Method

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A description is given of newly developed light equipment for vibration testing of road constructions. Sinusoidal loading is produced by means of an electrodynamic vibrator exerting vertical forces on the surface of the road or layer under test. The equipment, mounted on a special trailer, is extremely flexible for application to this type of test and permits reliable results to be obtained under a wide variety of conditions.

Two types of test have been carried out: in the impedance method, the ratio of force to resultant velocity is measured in amplitude and phase; in the wave propagation method, wavelengths along the surface are determined. Tests are carried out over a frequency range from 10 c/s to 500 c/s in the impedance method and up to 1,000 c/s in the wave propagation method.

Measurements have been carried out on an experimental road in which widely varying types of construction had been included. From the impedance measurements it was possible to ascribe dynamic parameters to each layer of the construction and, from these, stresses at various depths; and energy losses in the various layers have been calculated.

The potentialities of vibration testing in the control, design and evaluation of road constructions are briefly reviewed.

•THE impedance method of vibration testing of roads, in which an oscillatory force is applied to the road surface and the resultant velocity measured in amplitude and phase is, in principle, a powerful method of investigation in that complete information is gained at any frequency for which measurements are available. Unfortunately no exact theory has been developed up to the present time, because the problem is one of three-dimensional wave propagation in a layered construction, which has defied all attempts at exact mathematical treatment. Even if a mathematical solution were found at some future date, it must of necessity involve a large number of parameters (elastic moduli for each of the layers, thicknesses, etc.) and be of such a form as to render it of little value in its application to practical measurements.

In the present work an attempt has been made to interpret impedance measurements in a manner suggested by the experimental results themselves. With some assumptions and approximations, it is possible to suggest a model of certain elastic properties and geometric dimensions, and from it to estimate certain mechanical quantities which can be related to road performance. The model and all deductions therefrom are put forward as a tentative method of interpreting experimental data. Possibly this approach may lead more speedily to the designing of dynamic tests of practical value to the road engineer.

PRINCIPLE OF IMPEDANCE METHOD

The test consists of applying a vertical force to the surface and measuring both this force and the resultant velocity of the surface in amplitude and phase.

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The mechanical impedance, Z, of the construction under test is defined as the ratio of the force, F, exerted on it, to its resultant velocity, v; thus

$$Z = \frac{F}{v} \tag{1}$$

Z is a complex quantity containing a real part, R, and an imaginary part, X, the resistance and reactance, respectively, in which

$$Z = R + jX \tag{2}$$

and $j = \sqrt{-1}$.

Unless the system contains internal sources of mechanical energy, R must be positive; the reactance, X, may be positive or negative.

If θ is the phase angle defined by

$$\theta = \tan^{-1} \frac{X}{R} \tag{3}$$

and $|Z|$ is the amplitude of Z, the following relations hold:

$$|Z| \cos\theta = R \tag{4}$$

$$|Z| \sin\theta = X \tag{5}$$

$$Z^2 = R^2 + X^2 \tag{6}$$

Experimentally the R M S amplitudes of F and v are measured, so that $|Z|$ is calculated directly from Eq. 1. The phase angle, θ, is also measured, and the quantities R and X become then immediately available from Eqs. 4 and 5 at any frequency for which the measurements have been taken.

EQUIPMENT USED IN THE TESTS

Sinusoidal vertical forces are produced by an electrodynamic force generator bolted to the lower face of a cast iron cylinder weighing one ton. During tests, the cylinder is supported by two reinforced motorcycle innertubes fitting around its circumference and resting on circular brackets. The cylinder and its supports are mounted on a mo-

Figure 1. Station wagon and trailer set up for an impedance test.

bile trailer suspension system. An International station wagon is used to tow the trailer and house the associated electronic equipment. Figure 1 shows the station wagon and trailer set up for an impedance test.

Details of the equipment used for applying forces to the road surface are shown diagrammatically in Figure 2. Exhaustive tests have indicated that, in its present form, the equipment is suitable for impedance measurements covering the range from 10 c/s to 500 c/s. Although the forces generated are relatively low (not greater than 20 lb), they are sufficient to produce measurable velocities well above the background noises. Such forces can probably provide all the information obtainable from the application of larger forces, as road constructions have been found to be linearly elastic for forces up to 2 tons (1).

The electronic equipment is shown diagrammatically in Figure 3. It should be noted that both geophone and preamplifier must be calibrated for amplitude and phase over the whole frequency range to be investigated. The force output from the vibrator is monitored on the ammeter.

Experimental results for a particular site are most conveniently presented as two

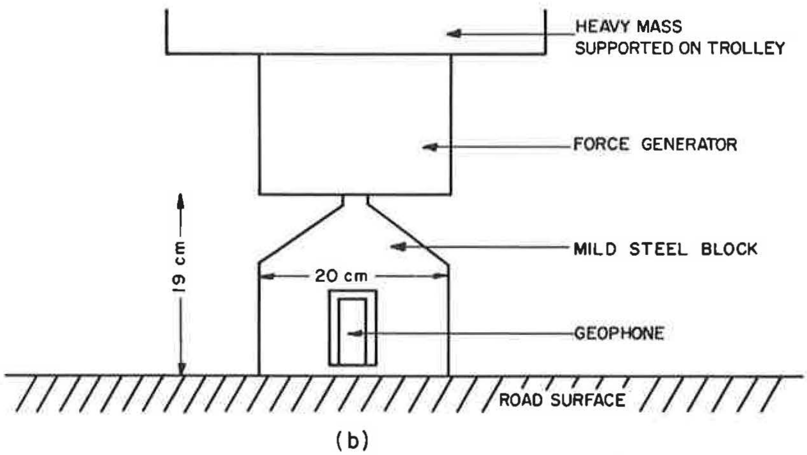


Figure 2. Framework through which forces are applied to surface.

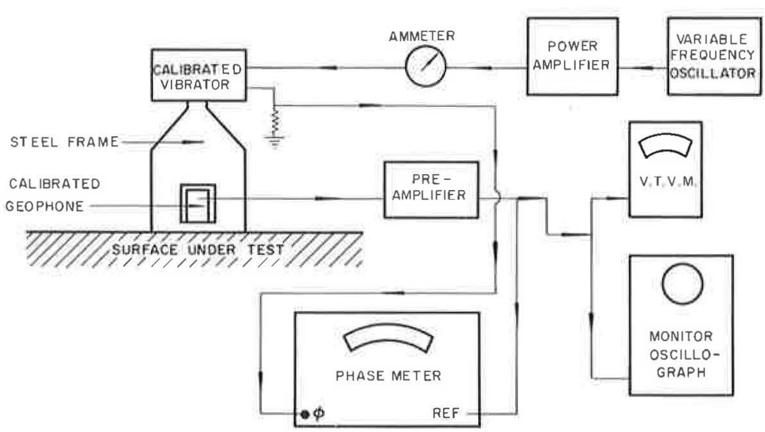


Figure 3. Block diagram of electronic equipment.

graphs relating X and R as functions of frequency. Typical curves are shown in Figures 4 and 5. It should be remarked that, except near resonance, ωX is numerically much larger than R and the phase angle, θ, is large; it therefore follows from Eqs. 4 and 5 that X can be determined more accurately than R.

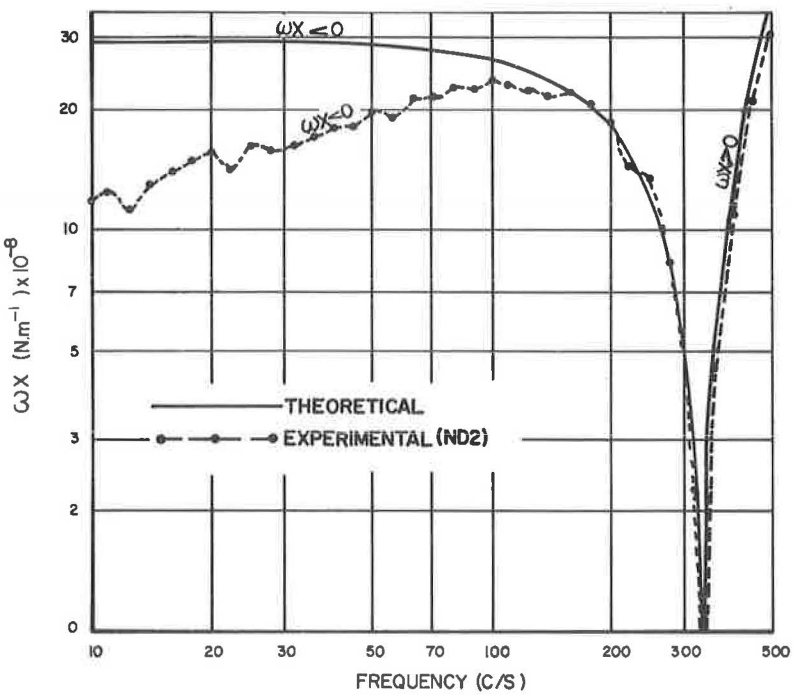


Figure 4. Typical ωX against frequency curve.

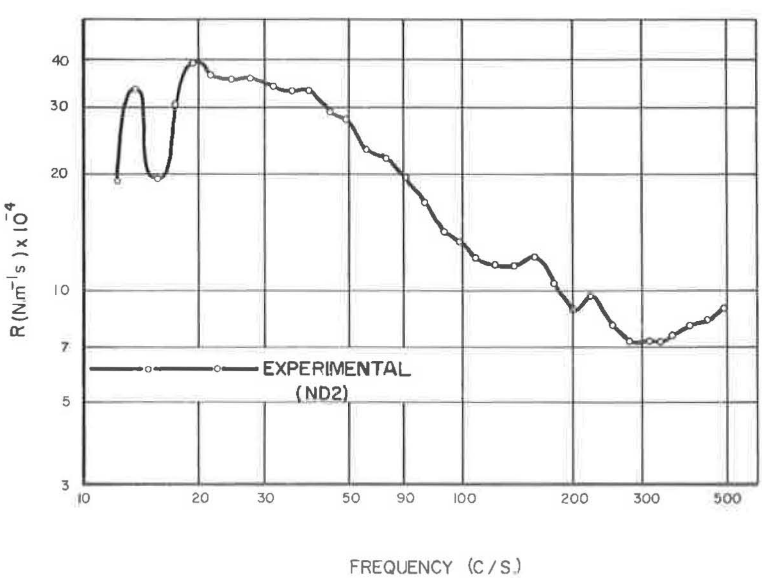


Figure 5. Typical R vs frequency curve.

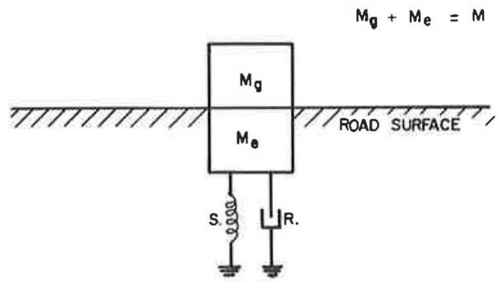


Figure 6. Assumed three-parameter model.

INTERPRETATION OF DATA FROM A MODEL WITH THREE CONSTANT PARAMETERS

The simplest mechanical model, for which the ωX function approximates in shape to the corresponding experimental curve, consists of a fixed M restrained in its motion by a spring of stiffness, S, and a frictional resistance, R (Fig. 6).

For this model the mechanical impedance, Z, offered to an impressed oscillatory force of frequency, f, is given by (2)

$$Z = R + j\left(\omega M - \frac{S}{\omega}\right) \qquad (7)$$

in which $\omega = 2\pi f$.

Identifying Eqs. 2 and 7, it follows immediately that: (a) the resistance, R, of Eq. 7 corresponds to the real part R of Eq. 2; and (b) the reactance, X, of Eq. 2 is related to the mass, M, and stiffness, S, of the adopted model by the relation:

$$X = \omega M - \frac{S}{\omega} \qquad (8)$$

or

$$\omega X = \omega^2 M - S \qquad (9)$$

A comparison between an experimental ωX curve and the corresponding theoretical curve obtained from Eq. 9 is shown in Figure 4, where the values of M and S for the theoretical curve were deduced from the experimental points around resonance (in which $\omega X \approx 0$).

The curves agree fairly well at higher frequencies, thus justifying the assumption of the three-parameter model. The greatest discrepancy occurs at low frequencies where the experimental curve is numerically lower than the theoretical one. The difference cannot be ignored, particularly as it occurs at all sites where the construction is of reasonably high quality. Furthermore, the R curve (Fig. 5), which should be constant with frequency, shows an appreciable drop. Hence it must be concluded that the parameters are not constant over the whole frequency range investigated, and the model with three frequency independent parameters is inadequate.

MORE COMPLETE INTERPRETATION OF EXPERIMENTAL DATA

It was suggested by Lorenz (3) that, for a three-parameter model of constant coefficients, ωX be plotted as a function of the square of the frequency, where a straight line should result; by putting $y = \omega X$ and $x = f^2$, Eq. 9 becomes

$$y = (4\pi^2 M)x - S \qquad (10)$$

The intercept on the y-axis is thus equal to -S and the gradient to $4\pi^2 M$; both M and S can therefore be determined, if these parameters are independent of frequency. Heukelom (4) in Holland and Baum (5) in Germany have adopted this approach and obtained some interesting results. Baum in particular, whose equipment operates between 10 and 70 c/s, has found that a plot of ωX vs f^2 falls along two straight lines, indicating two effective models each with its own constant parameters.

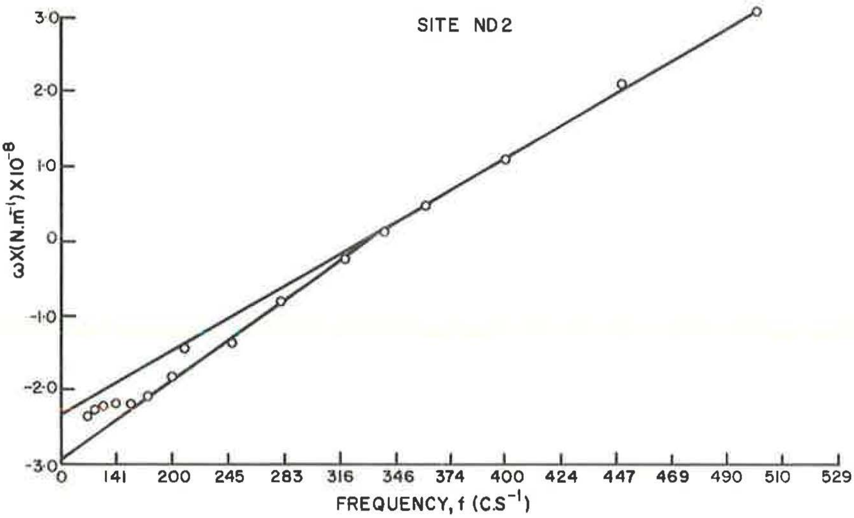


Figure 7. Plot of ωX vs f^2 for site ND2—high frequency range.

When this method is applied to measurements made by the authors it is found that, depending on the type of construction, one or two straight lines are obtained over the higher frequency range (Fig. 7) and sometimes even three lines. From this information the results given in Table 1 were compiled on the assumption that, in the frequency range where a number of successive experimental points lie on a straight line, the values of M and S are constant. According to Eq. 10 this is valid, unless S and M both vary in such a way that the variation of one exactly compensates for the variation of the other, which is most unlikely.

Straight lines on the ωX vs f^2 plot are obtained at the higher frequencies, usually above 100 c/s, depending on the type of construction. The fact that no use is made of the low frequency measurements is unsatisfactory in that some useful information must be contained in the data below 100 c/s.

In preliminary work it had been noted that, if S were assumed constant at low frequencies and equal in value to that obtained from the ωX vs f^2 straight lines, the experimental points followed quite well the theoretical values for M, obtained on the assumption that it varied according to

$$M = B f^{\alpha} \tag{11}$$

B and α being constants. B represents the mass when $f = 1$ and α was found to lie between -2 and -3. Experimental data therefore had indicated a very rapid increase in mass with decrease in frequency over the lower frequency range.

Physical considerations would also suggest a value of $\alpha = -3$, because, in a uniform medium with the wave velocity constant, the wave length, λ, is inversely proportional to the frequency.

$$\lambda \propto f^{-1} \tag{12}$$

Whatever the shape of the effective volume of disturbed material, its dimensions should be proportional to the wave length, and the volume to λ^3 or f^{-3}. Hence, the mass, M, would also be expected to vary as f^{-3}.

TABLE 1

VALUES OF S, M AND R DEDUCED FROM DATA SHOWN IN FIGURES 4 AND 5

Frequency (c/s)	S (N m^{-1})	M (kg)	R (N m^{-1}s)
100 - 140	2.54×10^8	506	1.20×10^5
160 - 320	2.95×10^8	67	8.97×10^4
340 - 500	2.43×10^8	56	7.88×10^4

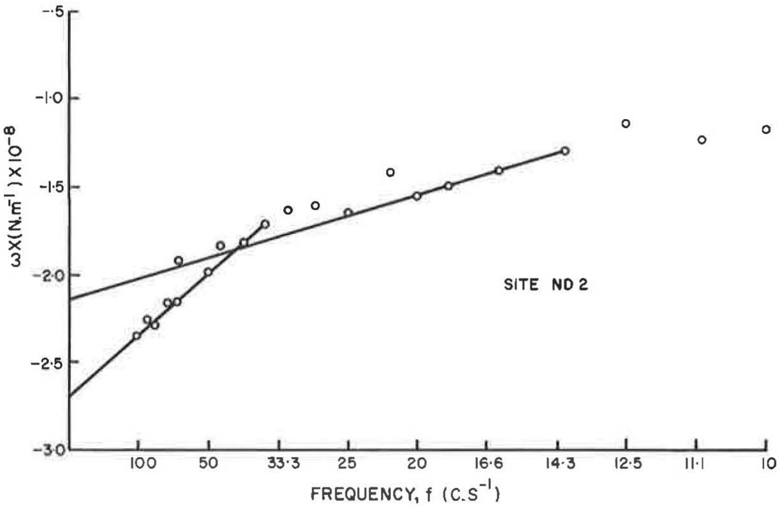


Figure 8. Plot of ωX vs f^{-1} for site ND2—low frequency range.

Therefore, it was assumed that the mass is given by

$$M = \pi B f^{-3} \tag{13}$$

Substituting Eq. 13 in Eq. 9 gives

$$X = (4\pi^2 B) f^{-1} - S \tag{14}$$

A plot of ωX vs 1/f should therefore give a straight line in a region where S is constant and M varies according to Eq. 13. Figure 8 shows a plot of ωX vs 1/f for one of the sites tested. Two straight lines could be drawn through the experimental points, from each of which values of S and B have been obtained (Table 2).

Using both the f^2 and f^{-1} plots it seems possible to classify the dynamic behavior of a road construction into a number of frequency ranges each having appropriate S and M values. Whereas the value of S is always constant in a given range, the value of M is only constant where its value is obtained from the f^2 plot. If the B-value is deduced from the ωX vs f^{-1} plot, the correct value of M at any frequency within the range of validity must be obtained from Eq. 13.

In view of the relatively larger scatter of the experimental points in Figure 5, a mean value of R for the frequency range, defined by the ωX straight-line relationships (Figs. 7 and 8), has been assumed. Values of R for site ND2 are given in the last columns of Tables 1 and 2.

DESCRIPTION OF EXPERIMENTAL SITES AND DATA

The measurements described here were made along an experimental length (approximately 1 mile) at Key Ridge on the southbound roadway of route N3/1 between Durban and Pietermaritzburg in Natal. The experimental length consists of 24 sections, each 200 ft long, and of different types of construction. Only 14

TABLE 2

VALUES OF S, B AND R OBTAINED FROM FIGURES 5 AND 2

Frequency (c/s)	S (N m^{-1})	B (kg s^{-3})	R (N m^{-1} s)
14 - 20	2.16×10^8	3.04×10^7	3.05×10^5
36 - 100	2.70×10^8	8.99×10^7	2.31×10^5

TABLE 3

EXPERIMENTAL VALUES OF THREE PARAMETERS FOR EACH FREQUENCY RANGE OBTAINED AT EACH POSITION OF SITE ND13

Position	1	2	3	Mean
Freq. range (c/s)	320 - 500	250 - 500	200 - 400	257 - 467
S (N m^{-1})	2.65×10^8	2.47×10^8	2.10×10^8	2.41×10^8
M (kg)	39	34.7	32.9	35.5
R (N m^{-1}s)	1.24×10^5	1.11×10^5	1.03×10^5	1.13×10^5
Freq. range (c/s)	80 - 125	80 - 160	80 - 200	80 - 162
S (N m^{-1})	2.46×10^8	2.08×10^8	1.82×10^8	2.12×10^8
B (kg s^{-3})	1.72×10^8	1.29×10^8	1.24×10^8	1.42×10^8
R (N m^{-1}s)	2.49×10^5	2.28×10^5	1.69×10^5	2.15×10^5
Freq. range (c/s)	45 - 64	50 - 71	40 - 71	45 - 69
S (N m^{-1})	1.89×10^8	1.66×10^8	1.30×10^8	1.62×10^8
B (kg s^{-3})	6.45×10^7	5.44×10^7	2.53×10^7	4.81×10^7
R (N m^{-1}s)	3.15×10^5	2.73×10^5	2.20×10^5	2.69×10^5
Freq. range (c/s)	20 - 28	16 - 40	10 - 40	15 - 36
S (N m^{-1})	1.41×10^8	1.31×10^8	1.12×10^8	1.28×10^8
B (kg s^{-3})	1.34×10^7	1.11×10^7	7.59×10^6	1.07×10^7
R (N m^{-1}s)	3.43×10^5	2.90×10^5	2.81×10^5	3.05×10^5

of the 24 sections were tested, profiles of which are shown schematically in Figures 9 through 12. All measurements were taken at a distance of 6 ft from the edge of the surfacing and three impedance tests were carried out on each section at 6-ft intervals. On section ND2, only one impedance test was made because of the steep gradient.

The data obtained on the various sites were analyzed as outlined in the previous section. Good agreement was found in regard to the number of straight lines for the three positions of each test site. A fair degree of variation was found in the parameters determined from the three sets of measurements taken on each section (Table 3). The mean values of the parameters obtained at the three sites on each section are given in Tables 4 to 7 in which the experimental sites have been grouped according to the type of construction, as follows: (a) group 1 (Fig. 9)—sites with various thicknesses of surfacing over "in-place" material; it should be noted that all these sites occurred in "cut," where the

TABLE 4

VALUES OF S OBTAINED BY THE METHOD FROM A MODEL WITH THREE CONSTANT PARAMETERS

Experimental Site	Frequency Range No. 1		Frequency Range No. 2		Frequency Range No. 3		Frequency Range No. 4		Frequency Range No. 5	
	Frequency (c/s)	S (N m^{-1})	Frequency (c/s)	S (N m^{-1})	Frequency (c/s)	S (N m^{-1})	Frequency (c/s)	S (N m^{-1})	Frequency (c/s)	S (N m^{-1})
ND5	10 - 23	1.36×10^8	27 - 67	1.73×10^8	160 - 310	2.07×10^8	333 - 500	1.53×10^8	-	-
ND8	83 - 147	1.01×10^8	273 - 450	8.50×10^7	-	-	-	-	-	-
ND9	17 - 35	1.35×10^8	107 - 200	1.38×10^8	333 - 500	6.67×10^7	-	-	-	-
ND10	10 - 32	1.35×10^8	28 - 53	1.38×10^8	120 - 200	1.55×10^8	320 - 500	1.41×10^8	-	-
ND15	15 - 47	1.17×10^8	64 - 97	1.57×10^8	247 - 500	1.90×10^8	-	-	-	-
ND2	14 - 20	2.16×10^8	36 - 100	2.70×10^8	100 - 140	2.54×10^8	160 - 320	2.95×10^8	340 - 500	2.43×10^8
ND6	10 - 16	1.28×10^9	20 - 40	1.54×10^8	51 - 97	2.13×10^8	220 - 303	2.27×10^8	347 - 500	2.11×10^8
ND7	15 - 39	2.36×10^8	68 - 140	3.83×10^8	347 - 483	4.52×10^8	-	-	-	-
ND11	16 - 45	1.64×10^8	65 - 173	2.80×10^8	333 - 500	3.48×10^8	-	-	-	-
ND12	17 - 38	1.79×10^8	44 - 93	2.16×10^8	167 - 287	2.45×10^8	340 - 500	2.40×10^8	-	-
ND3	29 - 42	3.66×10^8	50 - 113	3.61×10^8	180 - 265	7.25×10^8	280 - 385	4.06×10^8	410 - 500	2.70×10^8
ND4	17 - 27	3.83×10^0	42 - 64	5.40×10^8	66 - 110	7.50×10^8	147 - 297	1.07×10^9	353 - 460	2.50×10^9
ND13	15 - 36	1.28×10^8	45 - 69	1.62×10^8	80 - 162	2.12×10^8	257 - 467	2.41×10^8	420 - 500	1.65×10^8
ND14	13 - 37	1.77×10^8	45 - 128	2.75×10^8	180 - 260	3.26×10^8	-	-	-	-

TABLE 5

VALUES OF R OBTAINED BY THE METHOD FROM A MODEL WITH THREE CONSTANT PARAMETERS

Experimental Site	Frequency Range No. 1		Frequency Range No. 2		Frequency Range No. 3		Frequency Range No. 4		Frequency Range No. 5	
	Frequency (c/s)	R (N m^{-1}s)	Frequency (c/s)	R (N m^{-1}s)	Frequency (c/s)	R (N m^{-1}s)	Frequency (c/s)	R (N m^{-1}s)	Frequency (c/s)	R (N m^{-1}s)
ND5	10 - 23	2.82×10^5	27 - 67	2.45×10^5	160 - 310	9.60×10^4	333 - 500	9.01×10^4	-	-
ND8	83 - 147	3.37×10^5	273 - 450	2.94×10^5	-	-	-	-	-	-
ND9	17 - 35	2.03×10^5	107 - 200	1.35×10^5	333 - 500	1.25×10^5	-	-	-	-
ND10	10 - 32	3.12×10^5	28 - 53	2.41×10^5	120 - 200	1.66×10^5	320 - 500	1.52×10^5	-	-
ND15	15 - 47	2.23×10^5	67 - 97	1.67×10^5	247 - 500	7.60×10^4	-	-	-	-
ND2	14 - 20	3.05×10^5	36 - 100	2.31×10^5	100 - 140	1.20×10^5	160 - 320	8.97×10^4	340 - 500	7.88×10^4
ND6	10 - 16	3.10×10^5	20 - 40	2.94×10^5	51 - 97	1.91×10^5	220 - 303	6.21×10^4	347 - 500	3.23×10^4
ND7	15 - 39	4.69×10^5	68 - 140	3.11×10^5	347 - 483	8.93×10^4	-	-	-	-
ND11	16 - 45	3.30×10^5	65 - 173	2.32×10^5	333 - 500	6.52×10^4	-	-	-	-
ND12	17 - 36	3.33×10^5	44 - 93	1.69×10^5	167 - 287	7.46×10^4	340 - 500	5.06×10^4	-	-
ND3	29 - 42	6.45×10^5	50 - 113	4.57×10^5	180 - 265	4.46×10^5	280 - 385	1.54×10^5	410 - 500	1.56×10^5
ND4	17 - 27	1.03×10^6	42 - 64	1.11×10^6	66 - 110	1.08×10^6	147 - 297	9.14×10^5	353 - 460	7.76×10^5
ND13	15 - 36	3.05×10^5	45 - 69	2.69×10^5	80 - 162	2.15×10^5	257 - 467	1.13×10^5	420 - 500	8.09×10^4
ND14	13 - 37	4.34×10^5	45 - 128	3.12×10^5	180 - 260	1.73×10^5	-	-	-	-

TABLE 6

VALUES OF B OBTAINED BY THE METHOD FROM A MODEL WITH THREE CONSTANT PARAMETERS

Experimental Site	Frequency Range No. 1		Frequency Range No. 2		Frequency Range No. 3		Frequency Range No. 4		Frequency Range No. 5	
	Frequency (c/s)	B (kg s^{-3})	Frequency (c/s)	B (kg s^{-3})	Frequency (c/s)	B (kg s^{-3})	Frequency (c/s)	B (kg s^{-3})	Frequency (c/s)	B (kg s^{-3})
ND5	18 - 23	1.01×10^7	27 - 67	3.60×10^7	-	-	-	-	-	-
ND8	-	-	-	-	-	-	-	-	-	-
ND9	17 - 35	1.93×10^7	-	-	-	-	-	-	-	-
ND10	10 - 32	1.57×10^7	28 - 53	2.36×10^7	-	-	-	-	-	-
ND15	15 - 47	8.86×10^6	64 - 97	6.08×10^7	-	-	-	-	-	-
ND2	14 - 20	3.04×10^7	36 - 100	8.99×10^7	-	-	-	-	-	-
ND6	10 - 16	6.16×10^6	20 - 40	1.20×10^7	51 - 97	7.56×10^7	-	-	-	-
ND7	15 - 39	2.06×10^7	68 - 140	2.32×10^8	-	-	-	-	-	-
ND11	16 - 45	2.86×10^7	65 - 173	1.93×10^8	-	-	-	-	-	-
ND12	17 - 36	2.00×10^7	44 - 93	5.53×10^7	-	-	-	-	-	-
ND3	29 - 42	1.34×10^8	50 - 113	1.55×10^8	180 - 265	1.58×10^9	-	-	-	-
ND4	17 - 27	4.94×10^7	42 - 64	2.52×10^8	66 - 110	6.14×10^8	147 - 297	2.21×10^9	353 - 460	1.44×10^{10}
ND13	15 - 36	1.07×10^7	45 - 69	4.81×10^7	80 - 162	1.42×10^8	-	-	-	-
ND14	13 - 37	1.31×10^7	45 - 128	1.18×10^8	-	-	-	-	-	-

TABLE 7

VALUES OF M OBTAINED BY THE METHOD FROM A MODEL WITH THREE CONSTANT PARAMETERS

Experimental Site	Frequency Range No. 3		Frequency Range No. 4		Frequency Range No. 5	
	Frequency (c/s)	M (kg)	Frequency (c/s)	M (kg)	Frequency (c/s)	M (kg)
ND5	160 - 310	56.1	333 - 500	44.8	-	-
ND8	83 - 147	37.0	273 - 450	32.0	-	-
ND9	107 - 200	58.8	333 - 500	50.4	-	-
ND10	120 - 200	70.4	320 - 500	62.6	-	-
ND15	247 - 500	39.6	-	-	-	-
ND2	100 - 140	506	160 - 320	67.0	340 - 500	56.0
ND6	220 - 303	43.2	347 - 500	42.5	-	-
ND7	347 - 483	43.1	-	-	-	-
ND11	333 - 500	53.1	-	-	-	-
ND12	167 - 287	54.4	340 - 500	51.2	-	-
ND3	280 - 385	63.3	410 - 500	41.7	-	-
ND4	-	-	-	-	-	-
ND13	257 - 467	35.5	420 - 500	27.8	-	-
ND14	180 - 260	63.9	-	-	-	-

foundation might be expected to be above average strength; (b) group 2 (Fig. 10)—sites with foundation composed of base, subbase and subgrade, none of them stabilized; (c) group 3 (Fig. 11)—sites with a stabilized subbase; and (d) group 4 (Fig. 12)—sites with stabilized bases. All the sites of groups 2, 3 and 4 are on varying amounts of fill.

The total number of straight lines which could be found varies from a minimum of 2 to a maximum of 5; there also seems to be a slight tendency for this number to increase for constructions with stabilization either in the subbase or base.

The value of S always increases with frequency up to a broad maximum in the highest or second highest frequency range. On the other hand, R, decreases at various rates as the frequency increases.

From Eq. 13 and the values of B given in Table 6, it is seen that the decrease in M with increase in f is very rapid at low frequencies. At the high frequencies the mass remains constant over considerable ranges of frequency; however, the mass still decreases with increase in frequency in a number of steps.

The total mass, M, consists of the effective ground mass, M_e, which takes part in the vibration, and the mass, M_g, of the equipment below the force generator which also vibrates rigidly (Fig. 6). With the present equipment M_g = 34 kg, so that M can never be smaller than this value. Although the values of M obtainable from Table 6 over the low frequences are clearly much greater than this critical value, this condition is also

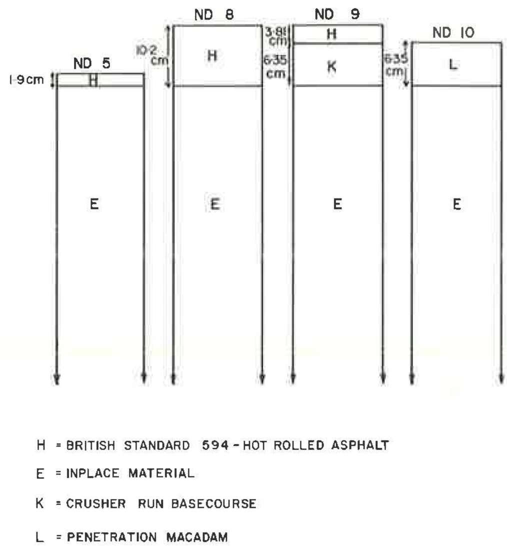


Figure 9. Profiles of test sites consisting of surfacing only.

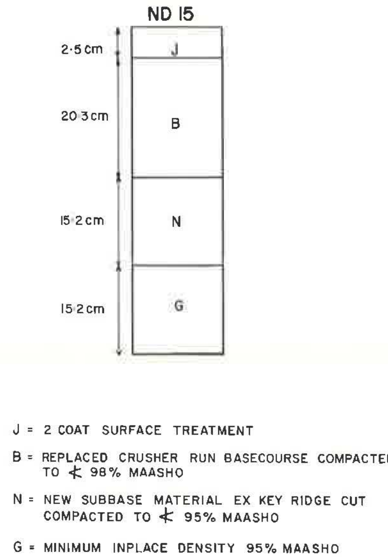


Figure 10. Profile of test site with unstabilized foundation.

satisfied in all cases shown in Table 7 except two (the second value for ND8 and the second value for ND13), where M is slightly smaller than 34 kg. The tendency of M towards this independently measured critical value confirms, therefore, the following points: (a) the part of the equipment below the force generator may indeed be considered rigid up to 500 c/s; (b) the calibration of all transducers and electronic equipment is sound; and (c) the measurements are, on the whole, reliable and have been correctly reduced up to this stage.

Although no mean values of M for each site can be taken on account of its rapid variation with frequency, some significance could be attached to the mean value of the other two parameters (S and R), as their variation with frequency is much smaller. The mean values of S and R for each site and each group are given in Table 8.

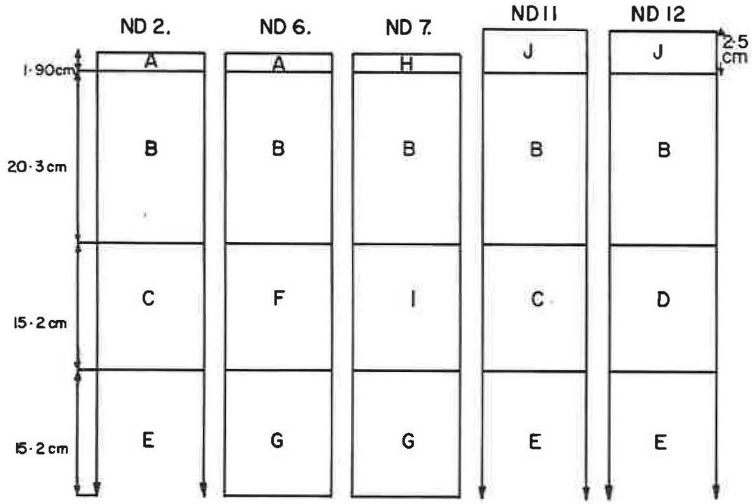
Considering the mean values for each group, it is noted that S is lowest for group 1 and highest for group 4. Group 4 also shows the largest value of R, while the lowest value occurs for group 2. However, there are considerable differences between the site values within each group, a fact not altogether surprising because the grouping was done in a rather arbitrary way as regards amount and type of stabilization. The high values for site ND4, which are presumably related to the 6 percent cement stabilization of the base, and the relatively low values for site ND13 where the base is stabilized with lime instead of cement, should be noted.

The differences between the means of each group are not much greater than the scatter within each group and therefore nothing further of significance can be deduced.

The stiffnesses, S, in Table 8 were compared with a few Benkelman beam results carried out on the same sites. An inverse correlation was noted between the stiffnesses found from impedance measurements and the deflections from the Benkelman beam tests, high values of S being usually associated with small deflections.

WAVE PROPAGATION TESTS

Another form of vibration testing makes use of the wave propagation method, in which the wave lengths of vertical surface waves are determined for various frequencies. The data are usually presented on a phase velocity vs wave length plot (Fig. 13) and interpreted in terms of a theory developed by Jones (6). For a layered construction, the dispersion graph results in a number of branches from which the sheer moduli and thicknesses of the layers can be determined accurately.



A = FLEXIBLE PREMIX.

B = REPLACED CRUSHER RUN BASECOURSE COMPACTED TO ≮ 98% MAASHO.

C = INPLACE MATERIAL COMPACTED TO ≮ 95% MAASHO AND STABILISED WITH 4% CEMENT.

D = INPLACE MATERIAL COMPACTED TO ≮ 95% MAASHO AND STABILISED WITH 6% MARVELLO WHITE ROAD LIME.

E = INPLACE MATERIAL.

F = NEW SUBBASE MATERIAL EX KEY RIDGE CUT COMPACTED TO ≮ 95% MAASHO STABILISED WITH 6% CEMENT.

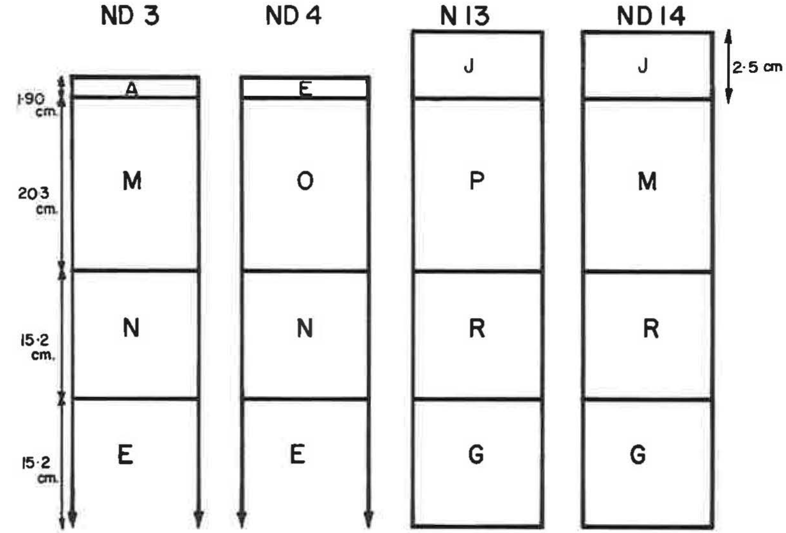
G = MINIMUM INPLACE DENSITY 95% MAASHO.

H = BRITISH STANDARD 594— HOT ROLLED ASPHALT.

I = NEW SUBBASE MATERIAL EX KEY RIDGE CUT COMPACTED TO ≮ 95% MAASHO STABILISED WITH 4% CEMENT.

J = 2 COAT SURFACE TREATMENT.

Figure 11. Profiles of test sites with stabilized subbase.



A = FLEXIBLE PREMIX.

M = REPLACED CRUSHER RUN BASECOURSE COMPACTED TO ≮ 98% MAASHO STABILISED WITH 4% CEMENT.

N = NEW SUBBASE MATERIAL EX KEY RIDGE CUT COMPACTED TO ≮ 95 % MAASHO.

E = INPLACE MATERIAL .

H = BRITISH STANDARD 594- HOT ROLLED ASPHALT.

O = REPLACED CRUSHER RUN BASECOURSE COMPACTED TO ≮ 98% MAASHO STABILISED WITH 6% CEMENT.

J = 2 COAT SURFACE TREATMENT.

P = REPLACED CRUSHER RUN BASECOURSE COMPACTED TO ≮ 98% MAASHO STABILISED WITH 6% MARVELLO WHITE ROAD LIME.

R = INPLACE MATERIAL COMPACTED TO ≮ 95% MAASHO.

G = MINIMUM INPLACE DENSITY 95% MAASHO.

Figure 12. Profiles of test sites with stabilized base.

TABLE 8

MEAN VALUES OF S AND R FOR EACH SITE AND FOR EACH GROUP

Group	Site	S (N m^{-1})	R (N m^{-1}s)
1	ND5	1.54×10^8	1.81×10^5
	ND8	9.30×10^7	3.15×10^5
	ND9	1.13×10^8	1.54×10^5
	ND10	1.38×10^8	2.18×10^5
	Mean	1.24×10^8	2.17×10^5
2	ND15	1.53×10^8	1.55×10^5
3	ND2	2.56×10^8	1.65×10^5
	ND6	1.87×10^8	1.78×10^5
	ND7	3.57×10^8	2.90×10^5
	ND11	2.64×10^8	2.09×10^5
	ND12	2.20×10^8	1.57×10^5
	Mean	2.57×10^8	2.00×10^5
4	ND3	4.26×10^8	3.72×10^5
	ND4	1.05×10^9	9.82×10^5
	ND13	1.82×10^8	1.97×10^5
	ND14	2.59×10^8	3.06×10^5
	Mean	4.79×10^8	4.64×10^5

Wave propagation tests were carried out on all the sites where impedance measurements had been made. The exciting forces were produced with the same equipment (vibration generator mounted under the trailer). Wave lengths were determined by measuring the increase in phase angle with increasing distance at various frequencies. The equipment was, unfortunately, unsuitable for measurements much above 1,000 c/s, with the result that: (a) the shear moduli of the surfacing could not be determined; (b) the shear moduli of the base course could only be determined approximately; and (c) thicknesses of the layers could not be calculated exactly from the measurements, but had to be estimated by assuming that the depth at which the elastic constants were determined was one-third of the wave length.

Profiles of the type shown in Figure 14 were calculated from the results obtained by this method and it was found that, in spite of the approximate method of determining depths, there was no difficulty in identifying the layers to which the shear moduli referred.

The thicknesses, although approximate, were used in later calculations to determine the geometry of the vibrating mass in the subbase and subgrade, where they were not always given in the design profile.

GEOMETRICAL SHAPE OF VIBRATING MASS

Values of the three parameters (mass, stiffness and resistance) cannot easily be related to the mechanical properties of road foundations with which the engineer is

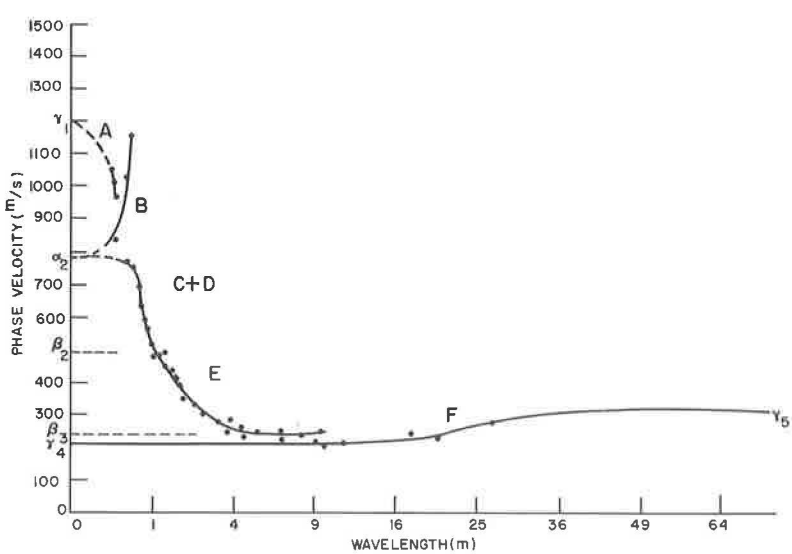


Figure 13. Phase velocity vs wave length graph for site ND 3.

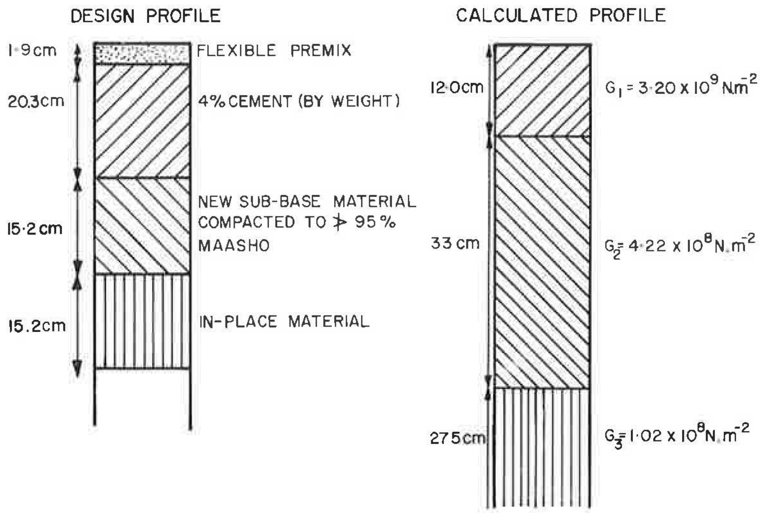


Figure 14. Design and calculated profiles for site ND 3.

familiar. On the other hand, impedance measurements are reasonably accurate and repeatable and, from their nature, should be usefully related to the mechanical properties of the constructions under test. The method of working with averaged values of S and R may give an overall picture of the construction, but what is required is more detailed information on the mechanical properties of each layer of the construction which would offer some indication of whether under-design or over-design has occurred.

More rational use must therefore be made of the measurements and their variation with frequency and, because of the lack of an adequate theory, a geometrical model is postulated which represents an approximation to actual conditions. In so doing the following observations have been borne in mind:

1. The three-parameter model implies a rigid mass of material in a state of vibration, restrained by frictional and elastic forces.
2. The vibrating mass varies rapidly with frequency, but in a somewhat irregular manner. At the higher frequencies it changes in a series of steps, while at the lower frequencies it varies continuously according to an inverse cube law as well as in steps as reflected in sudden changes in B (Eq. 13).
3. The number of steps is of the same order as the number of layers of the construction.
4. Both theory of, and experimental evidence from, the wave propagation method show that lower frequencies penetrate to greater depths of the construction.

These points suggest that a discontinuous increase in mass indicates that an additional layer has attached itself to the vibrating mass. If the layer is thin and bound together by strong internal forces, the transition from one layer to the next is represented by a sudden change in mass, while if the layer is thick and bound by weak internal forces, the increase in mass is continuous within the layer, while transition to a lower layer is accompanied by a change in B. If the thicknesses and densities of the various layers are known, it is possible to build a geometrical model using the variation in M obtained by impedance measurements.

The geometrical shape of the vibrating mass in each layer must be symmetrical in a horizontal plane, and static elastic theories also indicate that the load spreads in area as the depth increases; accordingly a frustum of a cone suggests itself for all the layers except the surfacing. To avoid unreasonably large spreads, which were found to occur for the surfacing, a cylinder has therefore been used to represent the shape of the top mass.

It does not necessarily follow that the number of frequency ranges must always be exactly equal to the number of layers in the road profile. Strong cohesive forces between two layers may give rise to a single frequency range (or M step). If a horizontal crack develops within one layer, or there occurs a horizontal boundary to the composition in a natural soil, the nominally single layer may give rise to more than one frequency range (or M step).

In the present series of measurements, the nominal thicknesses of the top layers were available from the design sheets for the experimental road supplied by the Department of Transport. The use of these values is not entirely satisfactory because the thicknesses of the actual construction may probably differ from those indicated in the design profile. A method based on wave propagation measurements is being developed by which it should be possible to determine thicknesses in situ to a fair degree of accuracy.

In practice, densities normally lie between 90 and 150 pcf and therefore represent a maximum variation of 25 percent about a mean. It will further be shown that a slightly incorrect density does not affect significantly the final results. Accordingly, for the determination of these models, a uniform density of 2×10^3 kg/cu m (equivalent to 120 pcf has been used.

In the following treatment the layers of the construction are numbered progressively from the top downwards, as follows: subscript 1 refers to the surfacing; subscript 2, to the base; subscript 3, to the subbase; subscript 4, to the subgrade. If M is the total mass obtained from the measurements at a given frequency and n layers are affected at that frequency

$$M = M_g + M_1 + \ldots = M_n \tag{15}$$

in which M_g is the apparent mass and $M_1 \ldots M_n$ are the layer masses taking part in the vibration. Because all the M's up to M_{n-1} are known from measurements at higher frequencies, M can be obtained by subtraction. In cases where M is a function of frequency, M has been calculated (Eq. 13) for the lowest frequency in the relevant frequency range. With these conditions in mind the following basic procedure was adopted to determine the geometrical shape of the models for each of the sites (Fig. 18):

1. The radius of the cylindrical mass associated with the surfacing was determined from the experimentally measured mass, M_1, and design height, h_1, from

$$r_1^2 = r_2^2 = \frac{M_1}{\pi h_1 \rho} \tag{16}$$

2. With r_2 determined, M_2 known from impedance measurements, and h_2 obtained from the design profile, a conical shape for the second block was assumed and its radius at the base, r_3, and the vertical angle, θ_2, determined from

$$r_3^2 + r_3 r_2 + r_2^2 = \frac{3 M_2}{\rho \pi h_2} \tag{17}$$

and

$$\tan \theta_2 = \frac{r_3 - r_2}{h_2} \tag{18}$$

3. The vertical angles of the lower blocks were obtained from the ratios of the elastic moduli of adjacent layers as determined from wave propagation experiments, according to

$$\frac{\tan \theta_n}{\tan \theta_{n+1}} = \left[\frac{E_n}{E_{n+1}}\right]^{1/2} = \left[\frac{G_n}{G_{n+1}}\right]^{1/2} \tag{19}$$

in which E_n and G_n are the Young's and shear moduli of the nth layer.

Eq. 19 (which assumes equal Poisson's ratios for adjacent layers) is strictly only applicable to the case of transmission of a plane compressional wave across the boundary between two elastic media, with no slipping at the boundary. However, its application here appears justified in that heights determined from Eq. 19 agreed well with design thicknesses where these were available.

4. Using the radius of the block above and the experimentally determined mass, the height, h_n, of the nth block was found from

$$h_n = \frac{\left[0.956 \frac{M_3}{\rho} \tan\theta_n + r_n^{\,3}\right]^{1/3} - r_n}{\tan\theta_n} \tag{20}$$

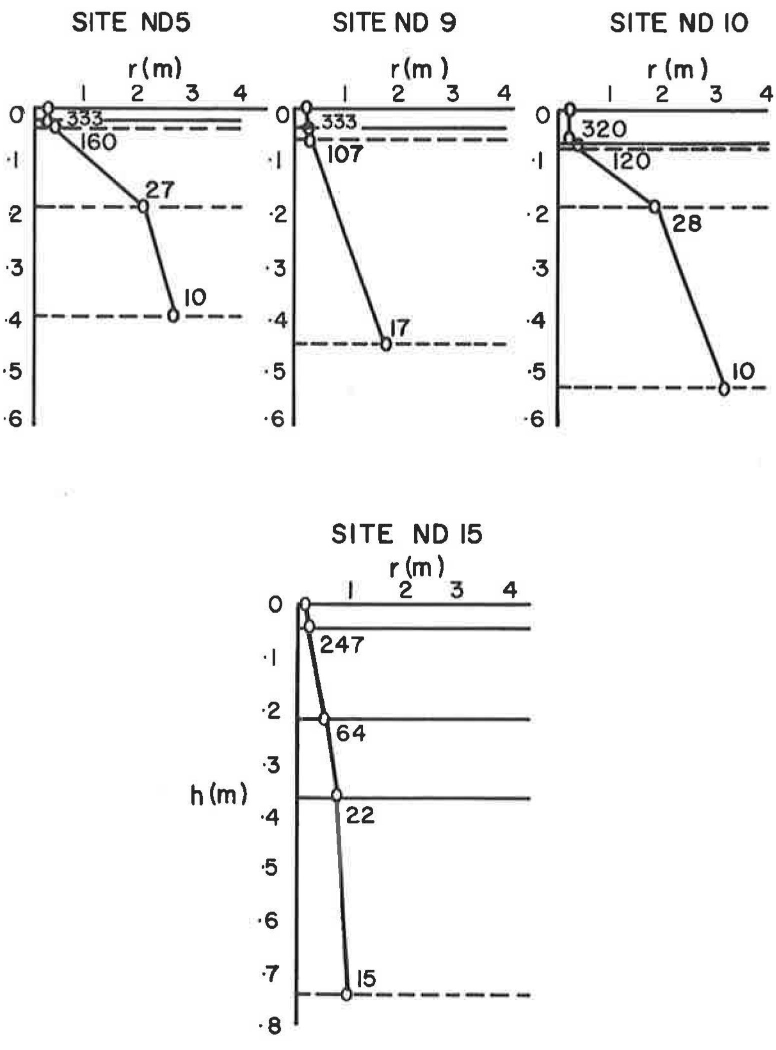


Figure 15. Variation of radii, r, with depth, h, for sites with unstabilized foundation.

and the radius of the base from

$$r_{n+1} = r_n + h_n \tan\theta_n \tag{21}$$

In some cases it was found necessary to combine more than one mass layer as explained earlier. A useful guide is provided by noting the frequency above which a block ceases to vibrate. For similar types of construction these limiting frequencies should be of the same order of magnitude at a given level.

The calculated variations of radii, r, with depth, h, are given in Figures 15 to 17, where separation between design layers is indicated by full horizontal lines, whereas the dashed lines show levels obtained by the method described under 3 and 4. It will be noted that the differences are usually small, and in some cases the design and calculated layer levels are practically coincident.

The numbers given at the layer separation points in Figures 15 to 17 refer to the frequencies above which the layers of the construction, below these points, are not involved in the vibration. No satisfactory model was obtained for ND8 because of lack of realistic masses.

In general, it may be remarked that the models for the sites such as ND15 (no stabilization) and ND13 (lime stabilization in the base) extend to greater depth than the

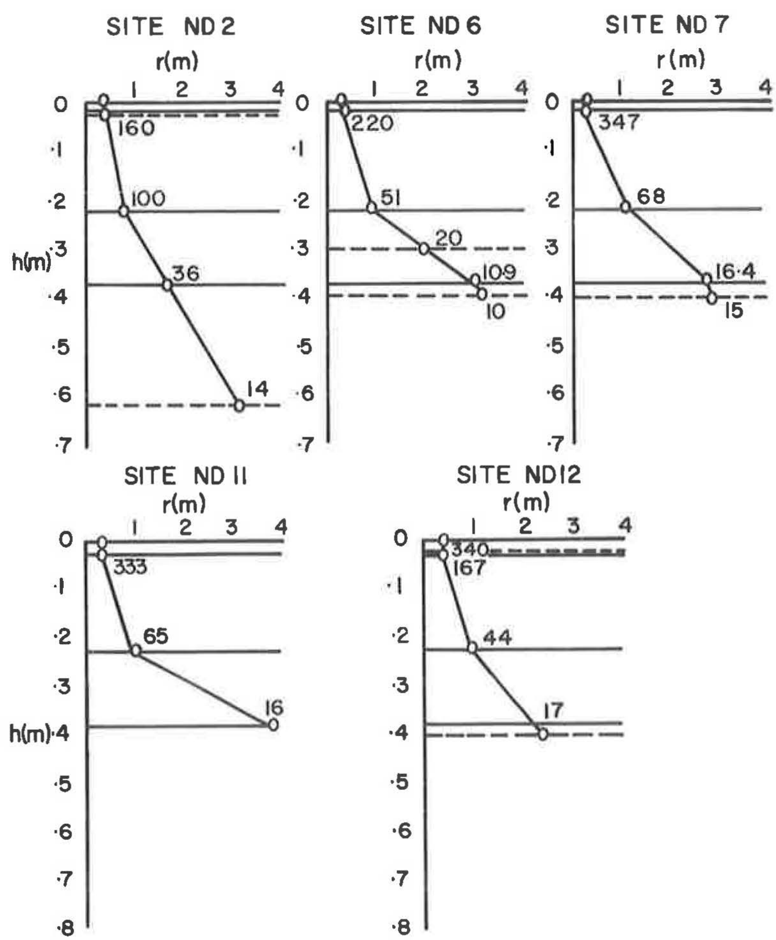


Figure 16. Variation of radii, r, with depth, h, for sites with stabilized subbase.

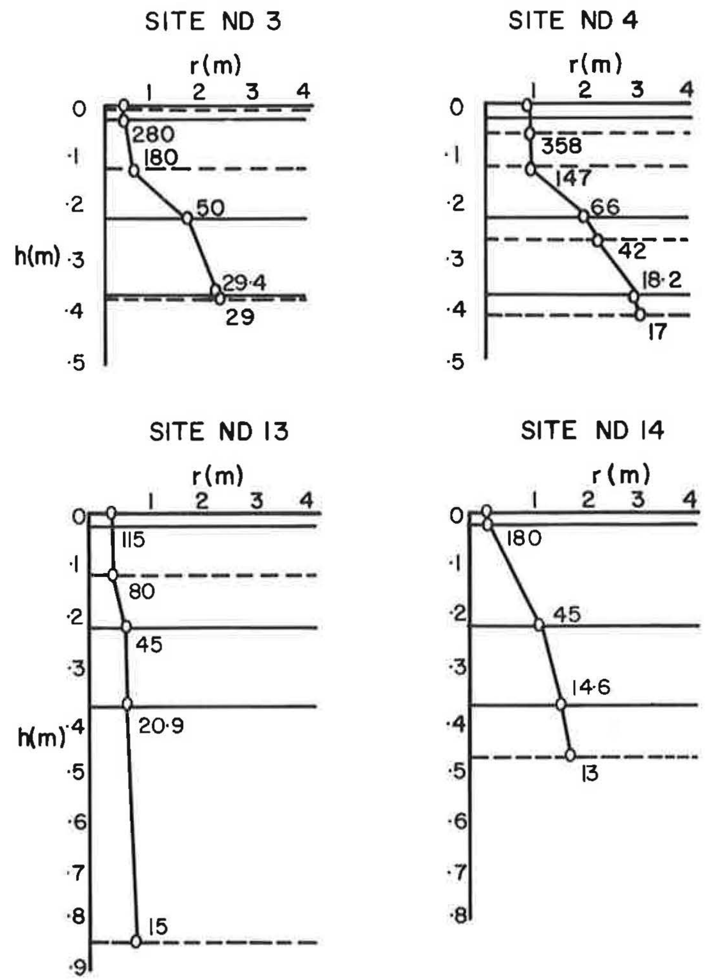


Figure 17. Variation of radii, r, with depth, h, for sites with stabilized subbase.

models for more solid constructions, such as ND3 and ND4 (with cement stabilization in the base). Furthermore, for most of the stabilized constructions (Figs. 14 and 15) (except for ND13), the model does not extend much below the subbase level, showing that forces at frequencies higher than 10 c/s do not affect the subgrade to any considerable extent.

ESTIMATION OF STIFFNESS AND RESISTANCE ASSOCIATED WITH EACH LAYER OF THE CONSTRUCTION

Stiffnesses and resistances obtained from impedance measurements (Tables 4 and 5) represent the total values of these quantities associated with that part of the model which is in a state of vibration at any particular frequency.

The mechanism of the model can be understood by reference to Figure 18, where it is shown how, for any layer, n, both stiffness, S_n, and resistance, R_n, have been divided into side components, S_{n1} and R_{n1}, and base components, S_{n2} and R_{n2} such that

$$\left.\begin{aligned} S_n &= S_{n1} + S_{n2} \\ \text{and} \quad R_n &= R_{n1} + R_{n2} \end{aligned}\right\} \qquad (22)$$

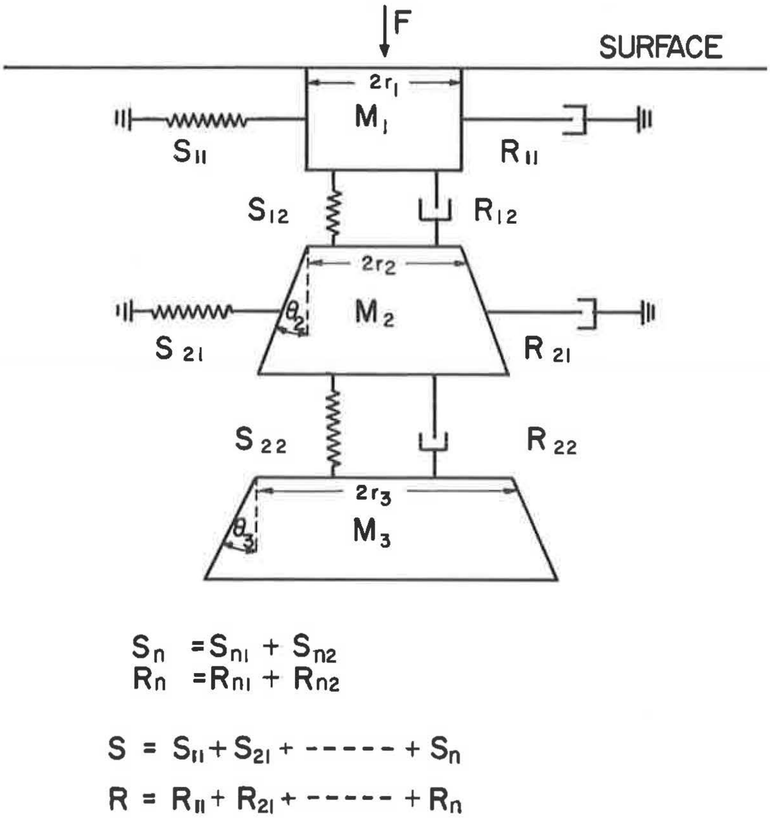


Figure 18. Diagram illustrating the division of stiffness, S, and resistance, R, into base and side components.

When, at a lower frequency, layer $(n+1)$ comes into action, it attaches itself rigidly to a layer n over the whole base area, so that S_{n2} and R_{n2} become inoperative, whereas S_{n1} and R_{n1} remain effective. Values of S_n and R_n may therefore be obtained for any particular layer from

$$\left.\begin{aligned} S &= S_{11} + S_{21} + \ldots\ldots + S_n \\ R &= R_{11} + R_{21} + \ldots\ldots + R_n \end{aligned}\right\} \qquad (23)$$

provided that values for the side components of higher layers are known. S and R are the experimentally measured values in the frequency range for which n layers are disturbed.

The problem resolves itself therefore into finding how S_n and R_n divide themselves into base and side components.

Inasmuch as the vibrating mass is considered rigid, stiffness and resistance effects are confined to the surface of the model. It is therefore assumed that frictional resistance and compressive stiffness are proportional to the surface area of the model.

Because the stiffness over the base is necessarily compressive, and the stiffness along the sides is partly compressive and partly tangential, this must be taken into account in the calculations.

If Poisson's ratio is assumed as $\mu = 0.5$ and s = compressional stiffness per unit area, then $s/3$ = shear stiffness per unit area. Consider an applied vertical force, F; then (Fig. 19).

The compressive component on slant face = $F \sin \theta$.

The tangential component on slant face = $F \cos \theta$.

The compressive stiffness = $s \sin \theta$ perpendicular to surface.

The shear stiffness = $s/3 \cos \theta$ parallel to surface.

The compressive stiffness along vertical = $s \sin^2 \theta$.

The shear stiffness along vertical = $s/3 \cos^2 \theta$.

Therefore, total vertical stiffness per unit area of curved surface is

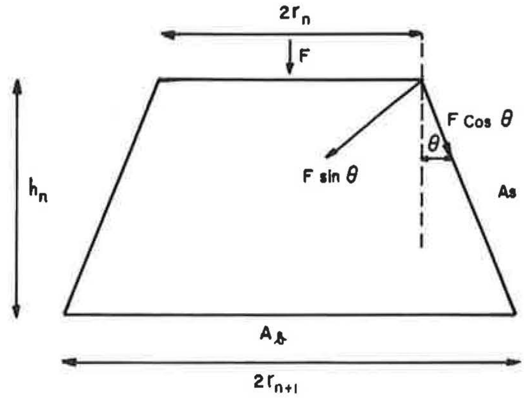
$$s (\sin^2 \theta + 1/3 \cos^2 \theta) =$$

$$s (1 - 2/3 \cos^2 \theta)$$

If A_s = area of curved surface and A_b = area of base,

the vertical stiffness due to the curved surface = $A_s\, s (1 - 2/3 \cos^2 \theta)$

and vertical stiffness due to base = $A_b s$



F = APPLIED VERTICAL FORCE.

A_s = AREA OF CURVED SURFACE.

A_b = AREA OF BASE.

θ = VERTICAL ANGLE OF CONE.

Figure 19. Determination of S_{n1} and S_{n2} from S_n.

$$\frac{\text{side stiffness}}{\text{total stiffness}} = \frac{S_{n1}}{S_n} = \frac{A_s (1 - \frac{2}{3} \cos^2 \theta)}{A_s + A_b (1 - \frac{2}{3} \cos^2 \theta)} \tag{24}$$

The resistance per unit area will be independent of direction if the tangential friction is equal to perpendicular friction so that

$$\frac{R_{n1}}{R_n} = \frac{A_s}{A_s + A_b} \tag{25}$$

A_s and A_b are fully defined by the geometry of the model and, in terms of the notation of Figure 19, are given by

$$A_b = \pi r^2_{n+1} \tag{26}$$

$$A_s = \pi (r_n + r_{n+1}) \sqrt{h_n^2 + (r_{n+1} - r_n)^2} \tag{27}$$

and for the case of a cylinder

$$A_s = 2 \pi r_n h_n \tag{28}$$

The division of S_n and R_n into their components depends, therefore, essentially on the ratio A_s/A_b. Consequently, with the thickness, h_n, fixed, the error introduced by a slightly incorrect model is greatly reduced, because both A_b and A_s vary in the same direction with a change in r_{n+1}, and a slightly incorrect value of ρ does not affect significantly their ratio.

Some of the results representative of each group are given in Figure 20 for S_n and in Figure 21 for R_n. The horizontal axis is proportional to the depth below the surface, whereas the heights are proportional to either S_n or R_n. The heights of the shaded areas are proportional to the side components (S_{n1} and R_{n1}, respectively), and the heights of the unshaded areas are proportional to the base components (S_{n2} and R_{n2}, respectively).

No direct comparison is possible, as all the sites tested represent different constructions. However, some general trends and characteristics are evident, as follows:

1. In general the value of S_n decreases as n increases, irrespective of the type of construction. An exception occurs for site ND3, where the maximum value of S_n occurs in the base course, which is cement-stabilized.

2. The value of S_{n1} (shaded areas) varies in a manner depending on the construction. In particular, the value of S_{11} is always lower than S_{21}, indicating that, as soon as the frequency is sufficiently low to excite the base, the surfacing contributes very little to the overall stiffness of the construction.

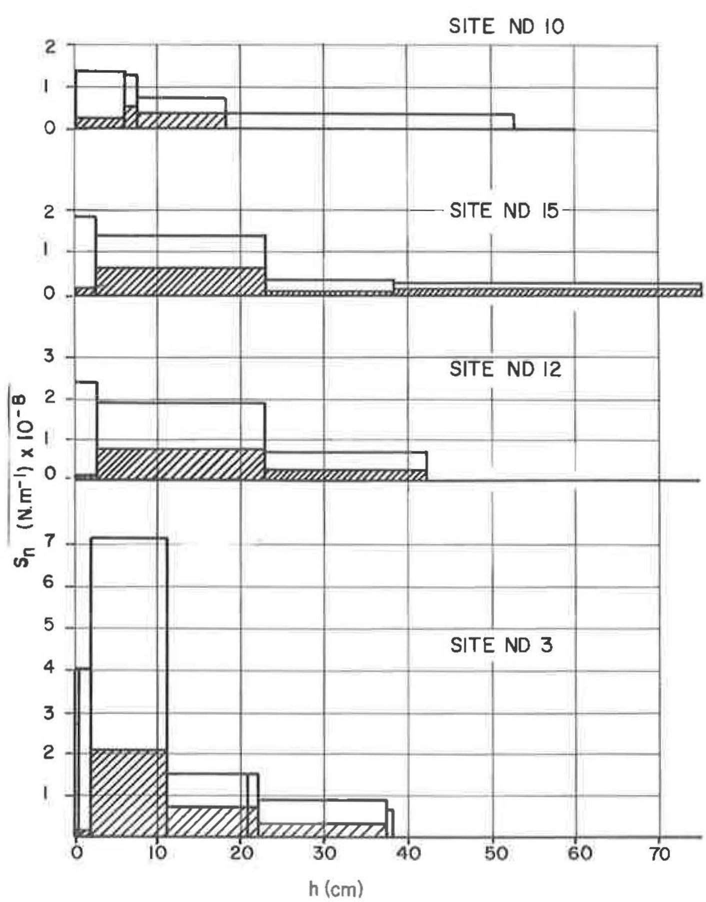


Figure 20. Comparison between S_n values for 4 types of constructions.

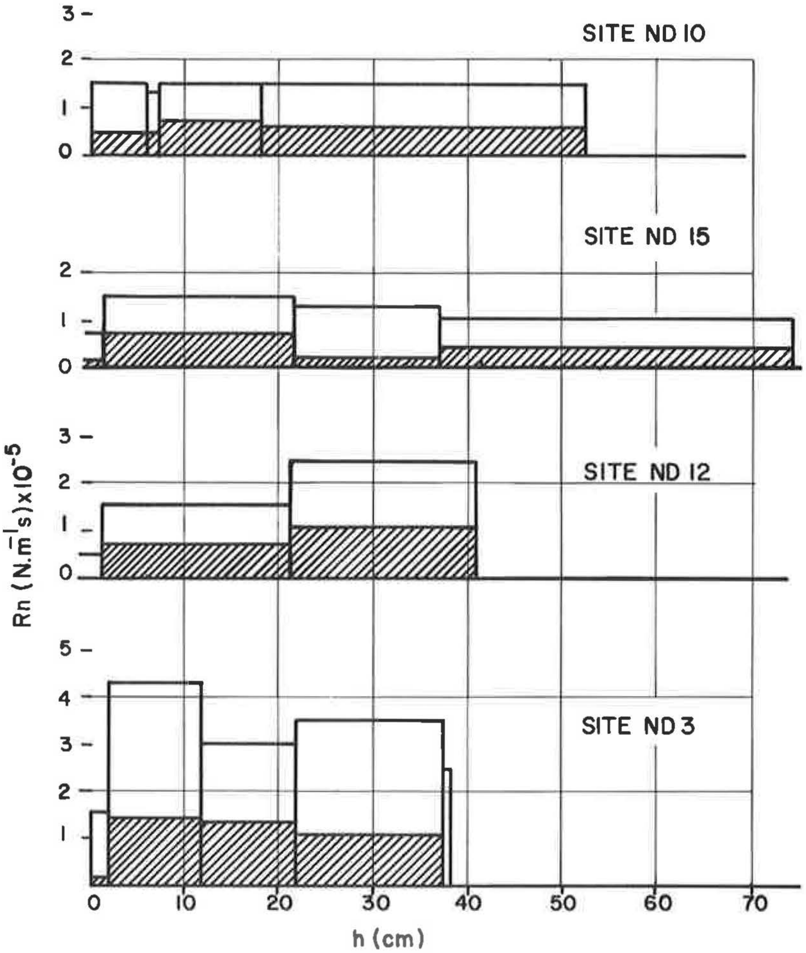


Figure 21. Comparison between R_n values for 4 types of constructions.

3. Large differences occur in the value S_1, S_2 and S_3 between the various constructions. The individual values cannot be taken as characteristic properties of the structure as a whole, but they represent the contribution of the layer under consideration to the overall stiffness of the construction.

4. Stiffnesses are higher for the better types of construction in Figure 20, corresponding stiffnesses being higher for the stabilized sites ND3 and ND12 than for the unstabilized sites ND10 and ND15.

Similar considerations are applicable to the R values given in Figure 21:

1. There is no definite trend in the R_n values, which vary to a much lesser extent with depth than do the S_n values.

2. The same is true of R_{n1} (shaded areas). Again R_{11} is always lower than R_{21}, indicating that the contribution of the surfacing to the overall resistance of the construction is negligible at lower frequencies.

3. Resistances are higher for the stabilized constructions (such as ND3 and ND12) than for the unstabilized ones (e.g., ND10 and ND15.)

DETERMINATION OF MECHANICAL QUANTITIES AT VARIOUS DEPTHS OF THE CONSTRUCTION

Once a suitable model has been found from impedance measurements and from it the division of S and R into their components for each layer estimated, other mechanical quantities at any depth of the construction may be estimated. The accuracy of such calculations will depend on the reliability of the original experimental data and on the degree of approximation involved in the various assumptions made in deriving the model which, it must be emphasized, is a simplification of actual conditions.

It is nevertheless felt that, if the same procedure is consistently followed in interpreting the measurements, comparison between results obtained from similar constructions should be relatively free of any errors introduced by the geometric dimensions of the model. In this and the following section, methods are developed for calculating the vertical stresses at any depth of the construction and the mean energy loss per cycle in a given layer at different frequencies of applied loading. Heukelom (4) has related frequency to equivalent vehicle speed, but this relationship has not been used here. It should be borne in mind that, although the values deduced here apply to sinusoidal loading, the passage of a vehicle wheel imposes a single impulse containing a large number of frequencies which are attenuated to different extents by the layers. The shape of such a surface impulse will therefore change with depth, whereas with sinusoidal loading, the amplitude only is reduced with depth, but not the shape.

Determination of Stresses

The adopted model consists of a rigid mass, M, vibrating with a vertical velocity, v, determined by the applied force, F, and mechanical impedance, Z, according to

$$v = \frac{F}{Z} \tag{29}$$

Therefore, with Z determined, v may be calculated for any impressed force, and the whole mass moves with this velocity.

The total force, F_n, acting at any level, h_n, of the construction over a horizontal area, A_n, will be

$$F_n = vZ_n \tag{30}$$

in which Z_n is the impedance of the model below level h_n. Z_n will be a function of frequency and in general is given by

$$Z_n = R' + j\,(\omega M' - \frac{S'}{\omega}) \tag{31}$$

in which R', M' and S' are the total values of R, M and S below the level, h_n, which are effective at the frequency $\omega/2\pi$. Because the model itself is a function of frequency it follows that, at a given level, Z_n is calculated by inserting the correct values of the primed parameters at the frequency under consideration.

The stress, σ, at level h_n and frequency f is then obtained from

$$\sigma = \frac{F_n}{A_n} \tag{32}$$

or

$$\sigma = \frac{vZ_n}{A_n} = \frac{F}{A}\,\frac{Z_n}{Z} \tag{33}$$

Inasmuch as both Z and Z_n are complex quantities, it follows that F_n and σ are not in phase with the applied force.

Stresses were calculated in this way for all the sites investigated at the levels of separation between layers. Because the area of cross-section of the cone increases with depth, the top of each cone section should in all probability give the value of maximum stress in the material.

The results for one of the representative sites are presented as curves of stress, σ, vs frequency at various depths in Figure 22. A wheel load of 1 ton (or 6.93×10^3 newton) was assumed, the load being spread evenly over a circular area of radius 10 cm (i.e., equal to the area used in the dynamic tests). This load corresponds to a surface stress of 2.20×10^5 N/m². Hence in all the graphs, σ has this value at all frequencies for h = 0.

The following points should be noted in Figure 22:

1. σ decreases rapidly as the depth h, necessitating the use of a logarithmic scale on the vertical axis.

2. At the higher frequencies (corresponding to higher vehicle speeds) only the higher layers are affected and hence the plot of σ for lower layers is very limited. This implies that, at high speeds, the surfacing and the base are of importance and subbase and subgrade are operative at low speeds only.

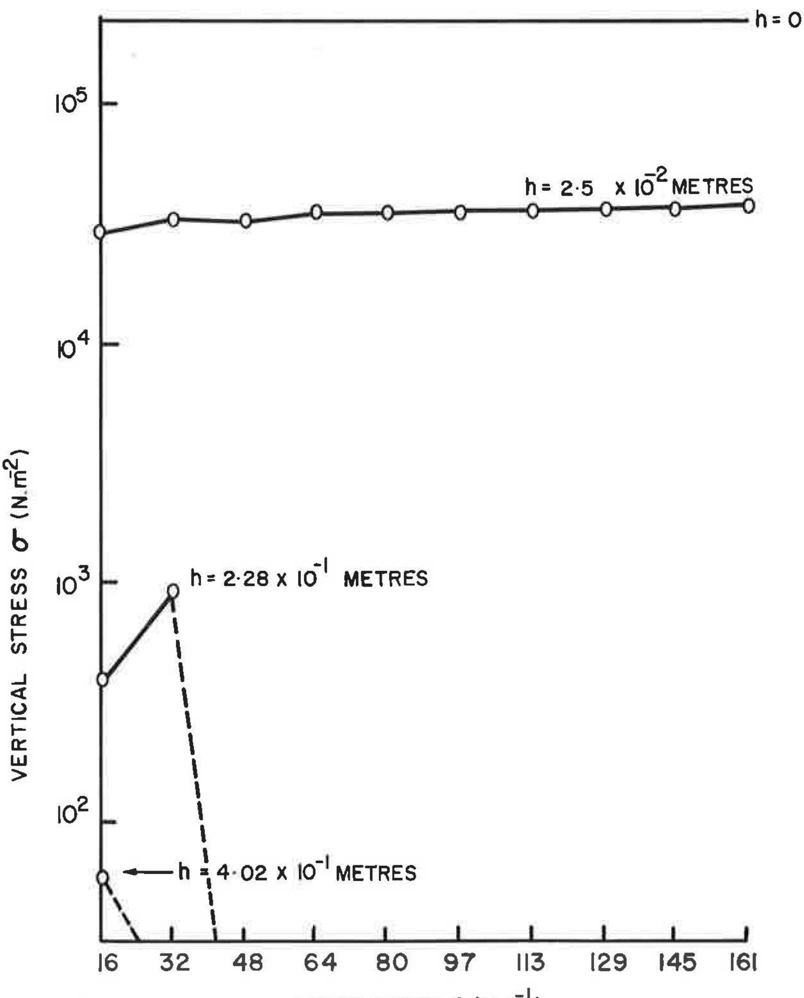


Figure 22. Variation of vertical stress, σ, with frequency, f, at various depths for site ND 12.

3. Graphs for the lower layers show a tendency to rise as f increases, showing that maximum stress occurs at the maximum frequency at which the layer is affected.

As σ varies comparatively little with frequency, its mean value may be used as an index of the reduction of stress with depth, independent of frequency. The results for some representative sites are shown in Figure 23, in which the heights of the lines are proportional to the logarithm of σ and their positions on the horizontal axis to the depth. The figures at the top of each line indicate the frequency below which the corresponding stresses exist.

The reduction of σ with depth is most marked for the sites with stabilized foundations (ND12 and ND3), but much less marked for ND15 where the foundation is unstabilized. The rapid drop in σ for site ND10 is remarkable and might be ascribed to the fact that the penetration macadam surfacing was apparently not well compacted.

Determination of Energy Loss

Another quantity which may be determined from the impedance method, after a geometrical model has been assumed, is the energy loss in unit volume. It is more

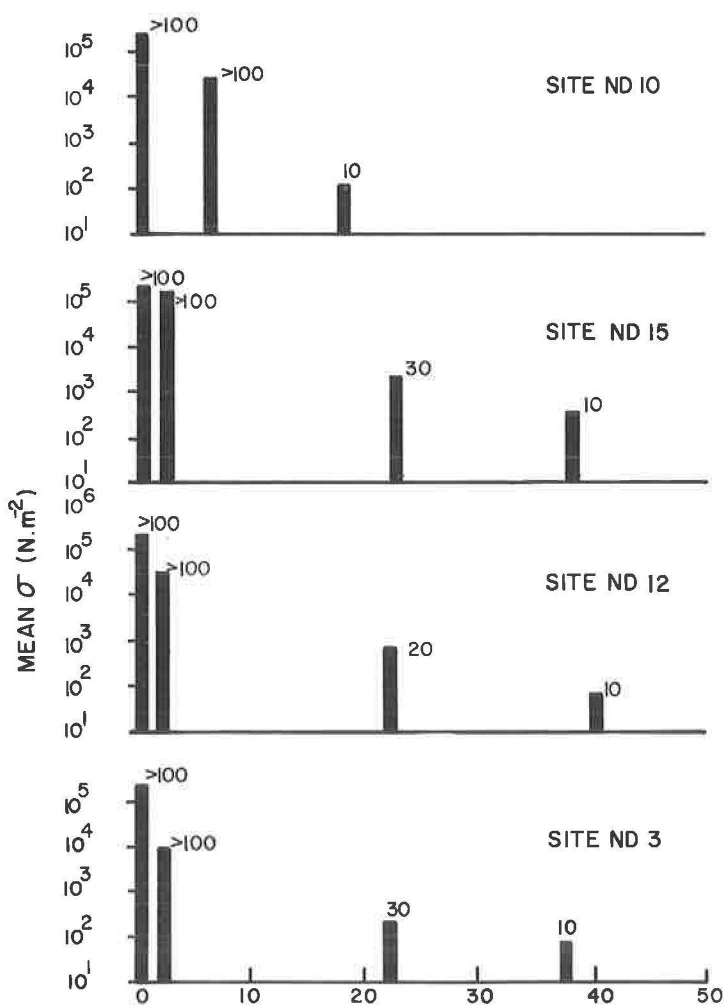


Figure 23. Mean vertical stress, σ, for 4 types of constructions at various depths.

useful to determine this quantity for one cycle, as in this form it may be more directly related to the passage of a single wheel load on the surface.

As before, it is assumed that the applied vertical force is sinusoidal and of frequency $f = \omega/2\pi$. Then the stress, σ_n, at a depth, h_n, below the surface will also be simple harmonic and of the form

$$\sigma_n = \sigma_o \exp(j\omega t) \tag{34}$$

and the vertical velocity, v (constant for the whole moving mass), is

$$v = \frac{F_n}{Z_n} = \frac{\sigma_n A_n}{Z_n} = \frac{\sigma_o A_n}{|Z_n|} \frac{\exp(j\omega t)}{\exp(j\theta_n)} \tag{35}$$

in which θ_n is the phase angle between σ_n and v, at depth h_n, i.e.,

$$\tan\theta_n = \frac{X_n}{R_n} \tag{36}$$

Hence

$$v = \frac{\sigma_o A_n}{|Z_n|} \cos(\omega t - \theta_n) \tag{37}$$

If p is the instantaneous power distributed throughout the vibrating volume, V_n, below h_n,

$$p = \vec{F}_n \vec{v} = A_n (\vec{\sigma}_n \vec{v}) = \frac{A_n^2 \sigma_o^2}{|Z_n|} \cos\omega t \cos(\omega t - \theta) \tag{38}$$

Thus the average power over volume below h_n is

$$\bar{p} = \frac{\int_o^T p \, dt}{T}$$

in which $T = 1/f =$ period of vibration.

Therefore, $$\bar{p} = \frac{\omega}{2\pi} \frac{A_n^2 \sigma_n^2}{Z_n} \int_0^{\frac{2\pi}{\omega}} \cos\omega t \cos(\omega t - \theta_n)\, dt = \frac{A_n^2 \sigma_o^2}{2|Z_n|} \cos\theta_n$$

Hence the energy lost per cycle in volume, V_n, below h_n is

$$E_n = \frac{A_n^2 \sigma_o^2}{2f\,|Z_n|} \cos\theta_n = \frac{A_n^2 \sigma_o^2}{f\,|Z_n|} \cos\theta_n$$

Expressing E_n in terms of v (see Eq. 33) gives

$$E_n = \frac{F_n \, v}{f} \cos \theta_n \tag{39}$$

If E_{n+1} is the energy lost per cycle in a vibrating volume, V_{n+1}, below some greater depth, h_{n+1}, and $E_n - E_{n+1}$ is the energy lost per cycle in the vibrating volume between depths h_n and h_{n+1},

$$E_n - E_{n+1} = \frac{v}{f}\left[F_n \cos \theta_n - F_{n+1} \cos \theta_{n+1}\right]$$

and finally, w, the energy lost per unit volume between levels h_n and h_{n+1} is

$$w = \frac{E_n - E_{n+1}}{V_n - V_{n+1}} = \frac{v}{f} \; \frac{F_n \cos \theta_n - F_{n+1} \cos \theta_{n+1}}{V_n - V_{n+1}} \tag{40}$$

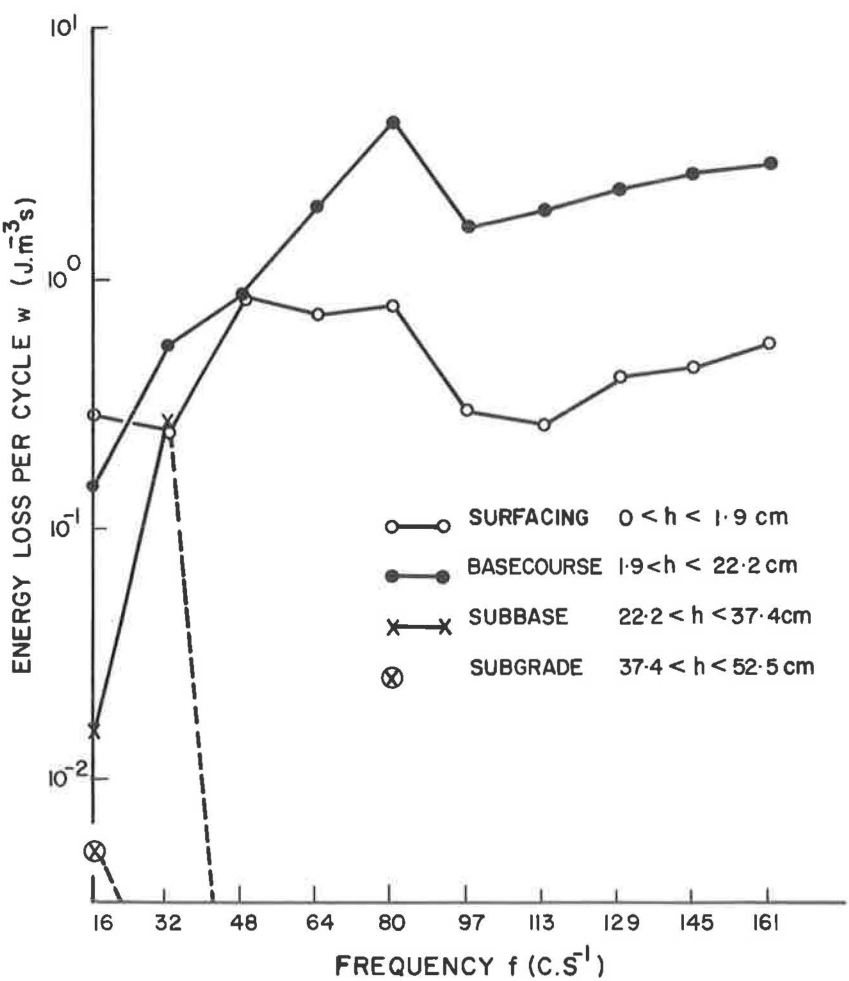


Figure 24. Variation of energy loss per unit volume per cycle, w, with frequency, f, for site ND 3.

in which the V's are calculated from the model's geometric dimensions.

Values of w at various levels obtained for site ND3 (as an example) are shown in Figure 24. In general, the graphs are characterized by a rising curve which reaches a maximum just before the layer under consideration becomes inoperative. Apart from the maximum, the energy loss is lower for deeper layers, but the latter show also a steeper gradient. The general characteristics shown in Figure 24 occurred also in the graphs for the other sites.

DISCUSSION

A method has been outlined by which a number of the mechanical properties of a layered construction can be determined quantitatively, almost entirely from impedance measurements.

No claim is made that the geometrical model adopted is unique or the most suitable, nor that further improvement in the method of determining its geometric dimensions cannot be made. However, the facts that the model must be symmetrical in a horizontal plane and that static theory indicates that the load spreads over a wider area with increase in depth, point unmistakably in the direction of a conical shape. There is a need for a more exact method of determining the radii of the cone frustums and this possibility is being investigated.

In spite of the uncertainty existing over the actual dimensions of the model, the general equations and methods of deduction of the more practical quantities (such as σ and w) should remain correct. Numerical values for the stresses are perhaps less reliable than those for the energy loss, in that the former are directly dependent on the square of the linear dimensions of the model, whereas the latter depend only on the volume (i.e., is independent of the radii of the cones).

Further vibration measurements on other roads are being carried out, as the 14 different types of construction tested are insufficient to enable conclusions of a general nature to be drawn; variations are, however, in the expected direction in most cases. A large number of measurements of nominally equivalent constructions are required to give more definite indication of the validity of the assumptions. Independent measurements of the derived quantities (e.g., the introduction of strain gages at various depths of a road construction, as is being done by Dempwolff, et al (7)), would be of great value, and are being planned.

The present work will be recognized as having its origin in the work of Baum (8). Progress, however, has been made in that it has been possible to assign values of the parameters to each layer of the construction. The remainder of the work follows almost axiomatically. An essential part of this development has been the wider frequency range of the equipment which has provided a more complete picture of mechanical behavior and without which no progress could have been made.

The present method of treatment can be criticized on the grounds that the problem has been oversimplified. The purpose, however, has been to attempt to bring impedance measurements one step nearer to the stage where they can become useful tools in the hands of the road engineer. The following applications can at present be envisaged:

1. Study of the deterioration of any road construction under traffic, by repeated measurements at regular intervals.
2. Survey and assessment of existing roads, and localization of required improvements.
3. Control of construction during road-building operations to check that each layer falls within required tolerances.
4. A simplification of rational design, based on elastic theory.

ACKNOWLEDGMENTS

The authors wish to thank C.H. Freeme and J. van Blerk who carried out the work in the field and Mrs. Y van Niekerk who assisted with the lengthy numerical computations.

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Discussion

E. N. THROWER, British Road Research Laboratory, Harmondsworth, England.— This paper presents a purely empirical discussion of the interpretation of dynamic stiffness tests on road constructions. It is based on a collection of arbitrary assumptions and ad hoc arguments which are difficult to discuss in a coherent fashion. Picking out just a few points:

1. It would have been somewhat less confusing if the results quoted by the authors for the impedance of a site had conformed to the definition given under "Principle of Impedance Method," where it is defined as the ratio between the total force exerted on the pavement itself, and its velocity. It is implied in the section on "Description of Experimental Sites and Data," however, that the results quoted refer to the ratio between the force applied to the vibrator foot and the pavement velocity; to obtain the impedance as originally defined the quoted values of ωX must be corrected by subtracting the reactance ωMg of the foot.

2. The arguments presented under "More Complete Interpretation of Experimental Data" assertion that the "mass" parameter will vary as f^{-3} does not involve the layered character of the construction. In fact, if true at all, it is true in particular for a semi-infinite uniform material. The calculations made by Sung (9), however, show that, on the contrary, ωX varies parabolically with frequency over the relevant part of the frequency range, so that the "effective mass" is constant. Thus there is no foundation for this argument. Further, if the argument were valid at all, it should be valid at all frequencies. Why then, is the dependence of the mass term (as distinct from the total ωX) taken as applying only at low frequencies? Lastly, the argument assumes that the phase velocity of the waves concerned is independent of frequency. Which of the many modes of vibration of a layered structure do the authors have in mind?

3. The assertion, implicit in "Description of Experimental Sites and Data," that M_e can never be negative, is also untrue; indeed, Figure 4 shows that at this site, $-\omega X$ increases with frequency, up to about 100 c/s, and this implies a negative value of M_e, as deduced from the plot of ωX against ω^2 used by the authors. This situation is avoided in the report by switching to an f^{-1} plot, but similar effects can also occur in the higher frequency range. Thus, if a sufficient degree of acoustic mismatch occurs between two adjacent

layers a resonant condition can arise from the reflection of longitudinal or shear waves at the interface. The resulting fluctuations in ωX cause negative values of M_e. Similar effects can arise from reflection of flexural waves at the edges of a road. Both these phenomena have frequently been observed in experiments made at the British Road Research Laboratory.

4. In connection with these points, perhaps the authors would indicate how their analysis would apply to the curves of Figure 25, which are calculated from the theoretical results of Warburton (10) and refer to the similar case of an elastic layer (shear modulus G, Poisson's ratio ¼) of thickness H, loaded over a circular region of radius r_0, and superimposed on an infinitely rigid substratum. The model thus assumes an infinite mismatch between the two media, and is to that extent unrealistic. Nevertheless, some sites approach this limiting case fairly closely, and the main effect of the finite mismatch (and of damping inherent in the materials) will only be to reduce the height of the resonant peaks. (The dotted portion of the curves denotes regions where the fluctuations in ωX are so rapid that the curves are not known accurately.)

5. Use of Eq. 19 which gives the angle of refraction of a plane longitudinal wave at the interface between two media, to calculate the angle of the fictitious cones of the author's model is without justification. Arbitrary introduction of this equation from the theory of an elastic continuum into a field to which it is completely irrelevant is unwarranted.

6. Although the elastic properties of bituminous materials vary rapidly with temperature, no reference is made to the effects of this parameter on the impedance values.

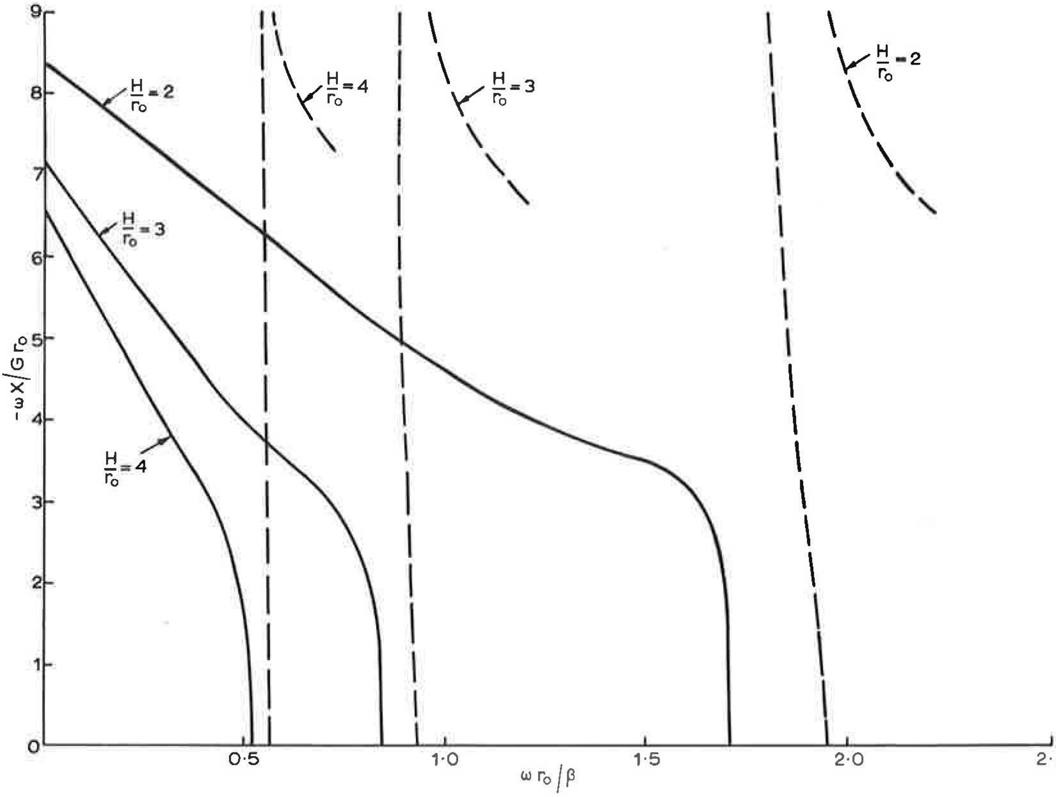

Figure 25. Dynamic stiffness of an elastic layer on a rigid subbase; after Warburton (10).

In general, analysis of impedance data along the lines of this paper, in which analysis and discussion of the real phenomena visible in the test results (e.g., thickness resonances, etc.) are replaced by the arbitrary introduction of as many ad hoc hypotheses as seem necessary, can ever lead to any fruitful results, especially when, as here, the hypotheses are unsupported, and indeed, often contradicted, by the available theoretical results.

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R. JONES, British Road Research Laboratory, Harmondsworth, England.—A few comments are offered here on the ambiguities and misconceptions which seem to exist in the section on "Wave Propagation Tests."

The writer's theory to which the authors refer is based on the assumption that the road may be treated as a plate (or composite plate) on liquid layers. These assumptions have allowed an interpretation of the experimental data obtained at the higher test frequencies, usually in the range of about 700 to 24,000 c/s, on major road constructions in Great Britain. The analysis permits the elastic moduli and thicknesses to be calculated of those cement- or bituminous-bound layers which have a considerably higher shear modulus of elasticity than the underlying soil or unbound granular layers. It is unfortunate that the apparatus described by the authors is not able to operate in that part of the frequency range in which alone a rational interpretation is believed to be possible at this time.

Absence of an adequate theory to interpret the low-frequency data does not, however, deter the authors from making confident identification of the depth and shear properties of the layers of a road by a process which has no rational justification. In data relating velocity to wave length obtained on our experimental roads we have been able to identify certain velocities which were constant over a range of low frequencies (long wave lengths) with the shear wave velocities in underlying granular materials. However, this inference was made only when the elastic properties of the granular layer had been previously measured during the road construction. As the authors are possibly aware, the depth of bases and subbases inferred from these results by the rule given in the paper (i.e., depth equals one-third of a wave length) was up to 7 times the true depth (11). Moreover, the depth found by the author's third rule can actually change with the date of test because the relations between wave length and velocity on a bituminous road change with the road temperature and, therefore, this causes changes in the complex modulus of elasticity of the bituminous materials. In the present state of knowledge, it is doubted that any region of constant velocity can be identified with the shear wave velocity of a particular layer without prior information. Why should not similar effects occur at compressional wave velocities, or even at velocities completely unconnected with the characteristic velocities in any of the layers as was found recently (12) in theoretical work on an earth model?

It is to be regretted that the authors have included so many apparently dubious assumptions and have, moreover, expressed confidence in their deductions without offering adequate proof or confirmation by means of independent measurements. This method of presenting controversial ideas is particularly regrettable because it can mislead newcomers to the subject and perhaps delay the development of a rational interpretation of the experimental data.

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M. E. SZENDREI and S. H. KÜHN, Closure—The authors wish to thank Dr. Jones and Mr. Thrower for their comments, which emphasize the present difference between two possible approaches in the investigation of dynamic testing. Although a number of theories have been developed, mostly based on idealized assumptions, little correlation has been established with practical road behavior. A number of other investigators, both in Europe and in the U.S.A., have on the other hand already applied dynamic tests, using approximate theories, with success in road and airfield design and the evaluation of performance under traffic.

In following the latter approach the authors admit that some of the assumptions made can be criticized theoretically. However, in the absence of an applicable theory and the evidence that results with practical significance are obtained, it is at present considered to be the best approach to investigate this promising method. The mechanical model used in the derivation of stresses and energy losses does not necessarily represent the only or best possible solution to the observed data. This fact has been mentioned in the introduction as well as at various stages in the text.

In reply to the comments by Mr. Thrower, the following are offered:

1. The fundamental definition of mechanical impedance, Z, defined as the complex ratio of applied force to resultant velocity, has been strictly adhered to throughout the text. Naturally it has been applied in each case with due regard to the mechanical model under consideration. Thus, all the experimental ωX values include the mass Mg of the vibrator foot, which under experimental conditions is an integral part of the vibrating system. When, however, in the latter part of the text, the net impedances of the construction are considered, Mg has been subtracted from the total mass measured.

2. The variation of the mass, Me, as f^{-3} at low frequencies is an experimental fact determined on different types of construction. Sung's theory, applying to a semi-infinite medium and assuming zero damping, therefore does not apply to the test conditions described. No theoretical explanation can be offered as to why the mass Me at high frequencies is constant, but this again follows from experimental results. The three-parameter model assumes a rigid mass in which the velocity is very high compared to the velocity in the surrounding medium.

3. The assumptions in regard to the variation of m with frequency have been consistently applied for all constructions analyzed and no cases were found where Me varies as f^{-3} in the high-frequency range. The authors feel that a negative Me is practically meaningless and that it is consequently justified to modify the assumed model to satisfy this condition at low frequencies. The authors do not agree that the experimentally determined decrease in ωX at low frequencies is due to acoustic mismatch, because this fact has been established on all the constructions tested, which included a variety of boundary conditions.

4. The suggested method of analysis could not be applied to the curves of Figure 25 drawn from the theoretical results of Warburton, inasmuch as curves of this shape have not been found to occur at the sites tested so far.

5. The application of Eq. 19 to the problem under discussion is admittedly of doubtful validity. It has been used only in extreme cases for the lower layers, when there was no independent means of determining the depth of the cone. However, as a check, it was applied also to a number of cases where the heights were known from the design profiles, and the resulting agreement was good as shown by the dashed lines of sites ND 6, ND 7 and ND 12 in Figure 16.

In reply to the comments by Dr. Jones, the following are offered:

The apparatus used in the wave propagation tests could operate satisfactorily up to a few kc/s. Higher frequencies, however, were not required to calculate the thickness of the surfacing-base combination, because it was determined entirely from the intersection of the first and second branches of the dispersion curves. The data required for the determination of λ were therefore the lower frequency points on the first branch and the higher frequency points of the second branch. The thickness of the surfacing was obtained from the design profile.

The other objection raised is the use of the empirical determination of depth assumed equal to one-third the wave length. Further work on this aspect since writing the paper has confirmed that approximate depths can be determined by the $\lambda / 3$ rule:—

1. A number of wave propagation tests carried out on roads with a cement-stabilized base and compacted subbase (of 3 times the base thickness) were analyzed using Jones' theory and the $\lambda / 3$ method. It was interesting to find that the differences in measured base thickness obtained by the two methods differed by no more than 5 percent.
2. Wave propagation tests were repeated on a road during various stages of construction. As each layer was added, the phase velocity/wave length dispersion curves were displaced along the wave length axis by an amount equal to about three times the thickness of the additional top layer.

According to a private communication received from the U.S.A., a theoretical derivation of layer depths using the wave length and the ratio of phase velocities in adjacent layers gave values varying between $\lambda / 2$ and $\lambda / 3$ and correlated well with independent direct measurements.

Some Recent Developments in Work On Skidding Problems at the Road Research Laboratory

C. G. GILES, British Road Research Laboratory, Harmondsworth, Middlesex

•WORK on skidding problems has been an interest of long standing for the British Road Research Laboratory. Early papers from the Laboratory drew attention to many questions which are still being written about and discussed today, such as the importance of surface texture, the different coefficient/speed relations on different wet surfaces and seasonal variations in slipperiness (1); calculations of braking distance from coefficient measurements, the importance of "rate of slip" and the difference between "peak" and "sliding" coefficients in braking force measurements (2); and the formulation of a specification for a reliable method of measuring and comparing the non-skid properties of road surfaces (3).

Much of this work has been written about and reviewed in more recent publications (4, 5), and the purpose of this paper is to give an account of the latest testing techniques which have been developed at the Laboratory and some of the research findings resulting from their use.

SKIDDING RESISTANCE MEASUREMENTS ON PUBLIC ROADS

Techniques for carrying out extensive programs of measurement on public roads are under consideration in some countries at the present time. Three recent developments in the Laboratory's work in this field are presented.

As is the practice in France, Belgium and Denmark, the sideway force method of test, using an inclined wheel, is the basic method employed by the Laboratory in making most measurements of skidding resistance on public roads (6). There are some good reasons for using this method of test. For example, as the test tire rotates continuously its wear is even, and there is little risk of results being influenced by localized heating or damage. A continuous recording is obtained, and essentially there is no limit to the length of road which can be covered in a single test. A constant rate of slip of the test wheel is obtained in a simple manner and the engine power required to maintain a given speed is appreciably less than that required to tow a locked-wheel trailer. Under British conditions it is also an important advantage that use of the method is not confined to straight, or nearly straight stretches of road. The test cars are so like ordinary cars in their appearance and handling that measurements can be readily made without interrupting or inconveniencing ordinary traffic.

Testing Resilience of Tires

To preserve continuity in the measurements, the testing conditions employed in the Laboratory's sideway force test cars are basically the same as those put forward by Bird and Scott (3). The test tires have always been specially made, and the Laboratory keeps the special mold used for this purpose. With the discovery of the importance of the hysteresis properties of tread rubber on friction on wet surfaces (7), it was suspected that steps would need to be taken to keep a check in some way on the resilience of all future test tires. Measurements showed that, in sideway force tests on wet

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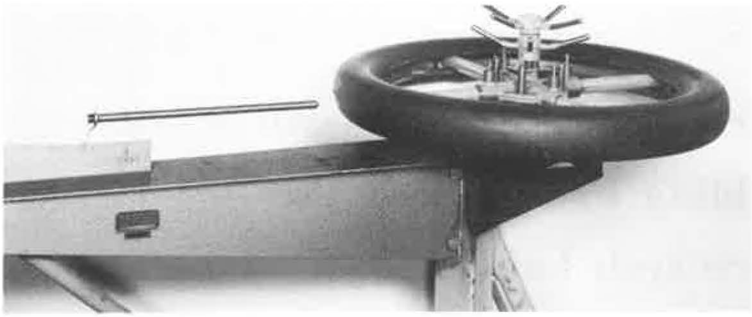

Figure 1. Lupke resiliometer modified for checking resilience of tires specially made for skidding resistance measurements.

coarse-textured surfaces, coefficient differences of the order of 0.10 at 30 mph could easily arise through differences in the resilience of the test tires.

Since 1960, measurements of resilience have therefore been carried out systematically on all the test tires used on the Laboratory's skidding machines. The apparatus used is shown in Figure 1. This consists of a Lupke resiliometer conforming to the requirements of British Standard No. 903 (8) which has been modified at the Laboratory to test tires instead of the more usual rubber discs. The resilience of the rubber is determined in terms of the percentage rebound of a hemispherically-ended steel rod supported on a long bifilar suspension which is allowed to strike the specimen under standard conditions. In the arrangement adopted, the test area of the tire is backed by a rigid metal "spoke" having a substantial head which is radiused to fit the inside profile of the tire casing so as to insure rigidity. A full range of spokes is available to suit all sizes of test tire which the Laboratory uses.

To indicate the need for this kind of testing, results of measurements on three batches of tires (B_1, B_2 + B_3) specially made for use with the Laboratory's small braking force trailer (9) and on two batches of tires for the sideway force machines (S_1 and S_2) are summarized in Table 1.

Batches B_1, B_2 and S_1 were made before the importance of resilience was recognized. In making the two subsequent batches (B_3 and S_2), close attention was paid to their resilience properties. The resulting improvement in uniformity is clearly evident, but even so there is a variation in the values of the order of ± 3 percent from the mean resilience of these batches, and there are also differences in the mean value from batch to batch.

On this evidence it would appear that, even where "standard" test tires are made in batches and conform to some closely specified rubber composition, differences in resilience between the different tire treads are to be expected. Some form of resilience test for tires is likely to be needed if good consistency in the measurements is to be maintained.

Evaluation of Test Results

Frequently, where the sideway force test method is used to test full-scale road experiments, a single set of measurements produces a paper chart containing in graphical form the results of up to five separate test runs at some chosen test speed over 100 or more experimental sections. The full evaluation of this record can be a time-consuming

TABLE 1

RESULTS OF RESILIENCE TESTS ON BATCHES OF SKIDDING TEST TIRES USING MODIFIED LUPKE RESILIOMETER

Batch	Range of Resilience Values in Batch (%)	Mean (%)
	(a) Braking Force Test Tires	
B1	37 - 54	46
B2	46 - 59	49
B3	54 - 59	57
	(b) Sideway Force Test Tires	
S1	27 - 53	44
S2	50 - 54	52

SECTION 7

RUN	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C
1	32	0.60	32	0.62	33	0.61	33	0.62	32	0.62	32	0.62
2	31	0.65	31	0.68	31	0.65	30	0.70	30	0.68	30	0.65
3	31	0.59	31	0.63	31	0.60	31	0.62	32	0.63	32	0.60
4	31	0.63	31	0.67	31	0.65	31	0.66	31	0.68	31	0.67
5	31	0.59	31	0.63	31	0.60	31	0.61	31	0.63	31	0.59

READINGS	MEAN SPEED	SIDEWAY FORCE COEFFICIENT MEAN	MAX	MIN	STD DEV
30	31	0.63	0.70	0.59	0.03

SECTION 8

RUN	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C	SPD	S.F.C
1	31	0.54	30	0.55	32	0.57	32	0.56	32	0.55	31	0.51
2	31	0.56	30	0.62	30	0.62	30	0.65	30	0.63	30	0.59
3	32	0.55	32	0.56	32	0.60	32	0.58	32	0.57	30	0.53
4	31	0.60	31	0.61	30	0.62	30	0.61	30	0.63	30	0.59
5	31	0.54	31	0.56	31	0.58	32	0.58	32	0.55	32	0.54

READINGS	MEAN SPEED	SIDEWAY FORCE COEFFICIENT MEAN	MAX	MIN	STD DEV
30	31	0.58	0.65	0.51	0.03

Figure 2. Example of tabulation and analysis of skidding measurements obtained using trace reader and Pegasus computer.

operation, and work is in hand to expedite this. At the present stage a chart analyzer has been developed which enables the recorded information to be transferred to punched paper tape. This can be fed into the Laboratory's "Pegasus" computer, and a program for this has been written to enable it to sort out and analyze the data automatically. An example of the way in which the results are finally arranged and printed is shown in Figure 2.

Transverse Positioning of Test Machines

Frequently, when the slipperiness of different road materials is under study, it is necessary to carry out skidding measurements at regular intervals and over quite long periods of time. Difficulties may then arise because of inevitable variations in skidding resistance due to uneven distribution of traffic across the width of the road and the difficulty of driving the test machine along precisely the same wheel track on every occasion. To try and meet these difficulties, advantage is being taken of other work being carried out at the Laboratory on electronic techniques for vehicle guidance and control (10). Field trials will soon begin with a system developed for guiding the test driver on to the correct position on the road. This system uses an energized guidance cable installed along the edge of the test road, and a single detector coil mounted on the bumper connected to comparatively simple electronic-indicating apparatus in the vehicle.

CROWTHORNE RESEARCH TRACK

Although full-scale experiments on busy public roads offer the most satisfactory means presently available of studying the skid-resisting properties of road materials under actual working conditions, it is becoming increasingly difficult to carry out many important researches under these conditions because of the higher speeds involved. The building of the Laboratory's Research Track at Crowthorne (11) has, therefore, provided an important opportunity for making some much-needed provisions to promote the Laboratory's work on skidding, braking and vehicle stability. The general layout of the track is shown in Figure 3, the parts specially concerned with this type of work being the Long Straight, the Terminal Area and the Central Area. On the Long Straight, six special test sections have been laid for skidding investigations, each 200 yd long by 12 ft wide, with provision for other test lengths to be laid as required. Three of the test sections represent surfaces of the rough coarse-textured kind which are commonly

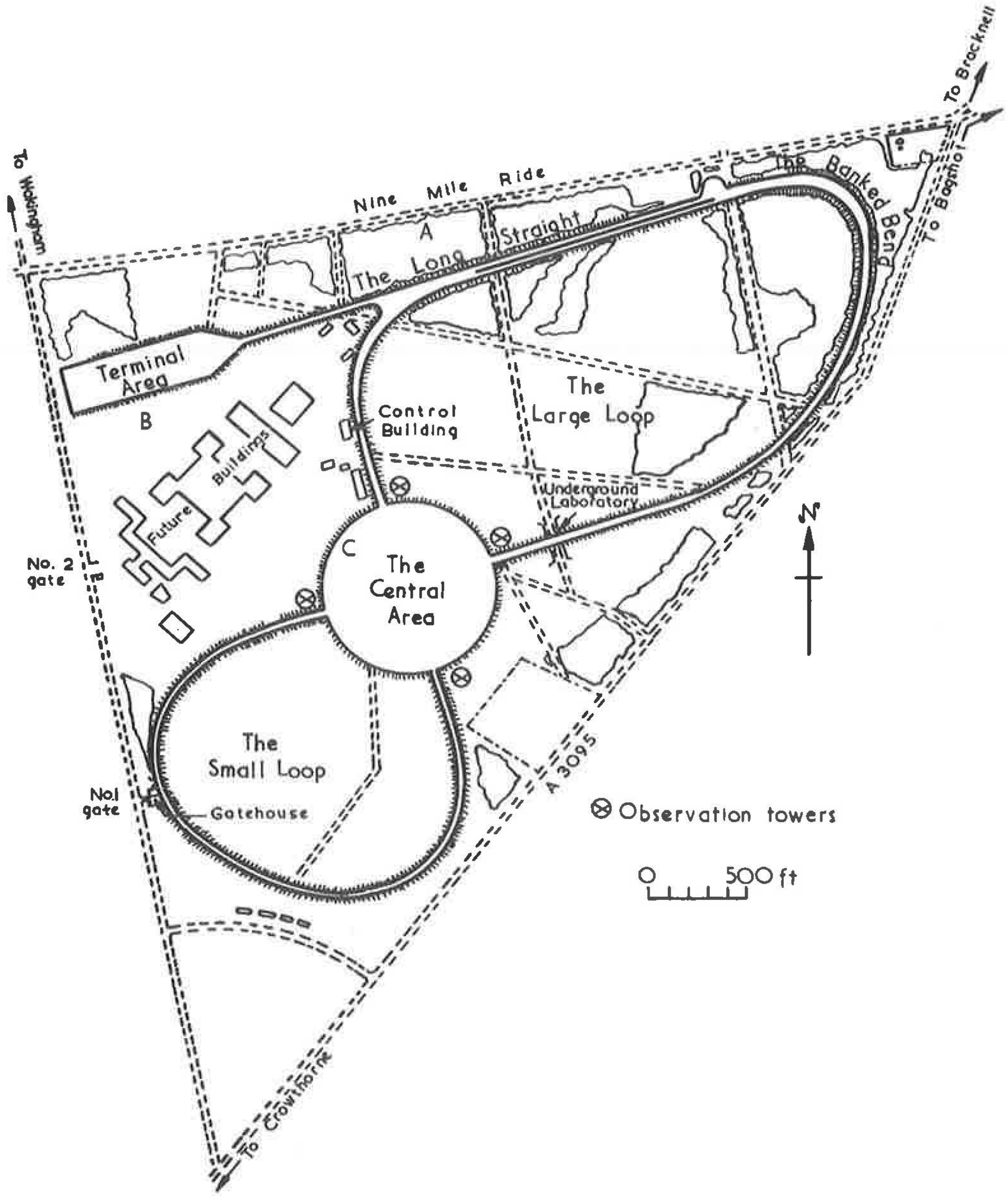

Figure 3. Crowthorne Research Track.

used on British roads, and three are surfaces of the smooth, or fine-textured type. In each section either the aggregate or the surface-finishing technique was specially chosen so that the three sections in each group cover a range of skidding-resistance properties from "non-skid" to very slippery. Skidding investigations are most frequently carried out on wetted surfaces, and the length of the Long Straight containing the six test sections has therefore been equipped with a watering system which is shown in operation at comparatively low pressure in Figure 4. The approach to the eastern end of the Long Straight is a curve of comparatively small radius (Fig. 3) and steeply banked. With suitable vehicles, approach speeds approximating 100 mph can be obtained. Stopping facilities at the other end of the Long Straight are provided by the Terminal Area which itself is 300 yd long and 83 yd wide.

Figure 4. Crowthorne Research Track spraying system in operation under comparatively low pressure.

Figure 5. Test trailer used for measurement of braking force coefficient.

MEASUREMENTS AT HIGH SPEED WITH SMALL BRAKING FORCE TRAILER

The Laboratory has been making measurements on aerodrome runways and roads with a small braking force trailer (9) (Fig. 5). Some typical results showing the different kinds of coefficient/speed relations which are found are shown in Figure 6. Smooth or fine-textured surfaces are represented by the steeper coefficient/speed curves shown in Figure 6, and coarse-textured surfaces by the flatter curves. For the test conditions used on the trailer the "tire-hydroplaning speed" using Horne's simple approximate formula (12) is only of the order of 47 mph.

With the completion of the Crowthorne track the coefficient/speed relationship for locked wheels on wet coarse-textured surfaces could be studied at higher speeds than

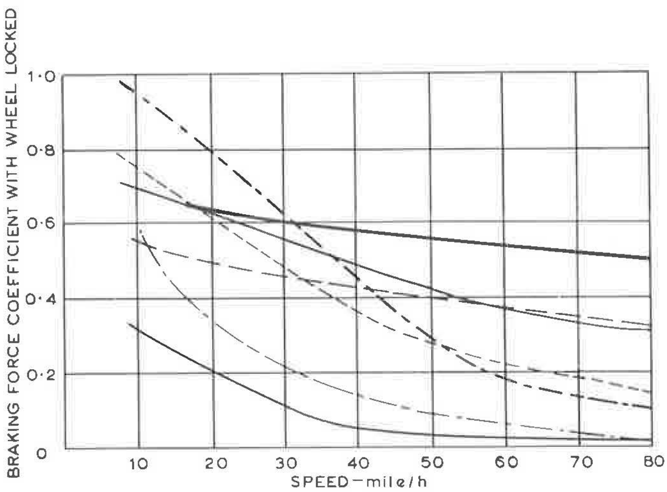

Figure 6. Results of braking force measurements on different wet surfaces made with small braking force trailer.

had previously proved possible. Results were rather unexpected as illustrated by the example shown in Figure 7a. This compares results obtained on the most "non-skid" of the smooth looking surfaces on the track (Fig. 7b) with results on the most slippery of the rough coarse-textured ones (Fig. 7c). On the latter surface at speeds above 80 mph the locked-wheel coefficients show a sharp increase in value. Similar increases in the coefficients above this speed have been obtained on the other rough coarse-textured surfaces on the track.

There are a number of clues to the reasons for this behavior. First, although the surfaces were maintained in a thoroughly wet condition, there was a smell of hot rubber each time the wheel was locked in the tests above 80 mph, and the tire showed evidence of a very deep "scalding" over the area actually in contact with the surface. This scalding appears to be due to heating and softening of the rubber just below the surface of the tread, by the energy dissipated there through hysteresis losses due to the repeated deformation and recovery of the rubber as the projections in the road surface are dragged through it. The speed of sliding appears to be an important factor because in tests made on similar surfaces using $^3/_{16}$- in place of $^3/_8$-in. stones, it was again necessary to exceed 80 mph before the rising coefficient effects could be observed. There may possibly be some connection between this and the rate at which deformations recover in natural rubber treads. For example, the sliding speeds are approaching the speeds at which "standing wave" effects in tires are observed (13), and there is also other evidence of retraction speeds of this order (14). Grosch (15) has discussed the effect of rate of recovery on rubber friction. Finally, when similar tests were made with a low resilience tire of butyl rubber, higher coefficients were obtained at all speeds, and similar tire damage occurred at very much lower speeds of test.

TABLE 2

EFFECT OF WATER DEPTH AND SPEED ON COEFFICIENTS OBTAINED WITH SMALL TRAILER APPARATUS ON WET SURFACES

Test Speed (mph)	Braking Force Coefficient[1]	
	Water Depth	
	0.007 in.	0.030 in.
26	0.70	0.69
60	0.20	0.11

[1] Small trailer apparatus, locked wheel.

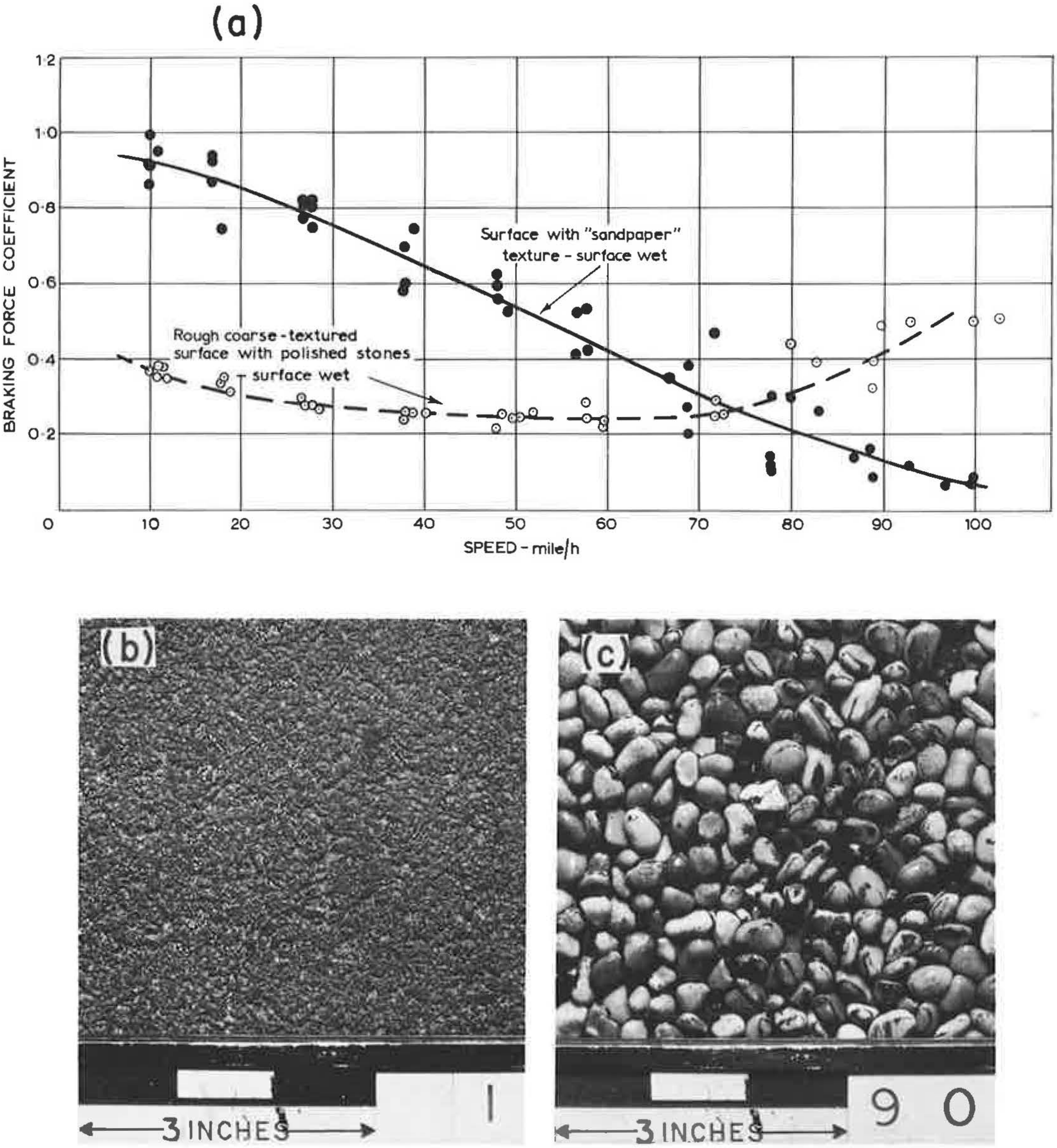

Figure 7. (a) Measurements of braking force coefficient obtained with locked wheels and patterned tires on two surfaces; (b) fine-textured asphalt test surface representing a "smooth-looking" surface with a good sandpaper texture; and (c) rough coarse-textured test surface made with highly polished stones to give a relatively low resistance to skidding when wet.

WATER DEPTH AND SKIDDING RESISTANCE

One factor relevant to skidding resistance which has been shown in the research work carried out in this field by the N.A.S.A. (12, 16, 17) is the importance of the inertia of the water when the tire is required to displace it at high speeds. Test results with the small trailer machine illustrating the effect of some comparatively small differences in water depth on a smooth surface and the importance of the speed are given in Table 2.

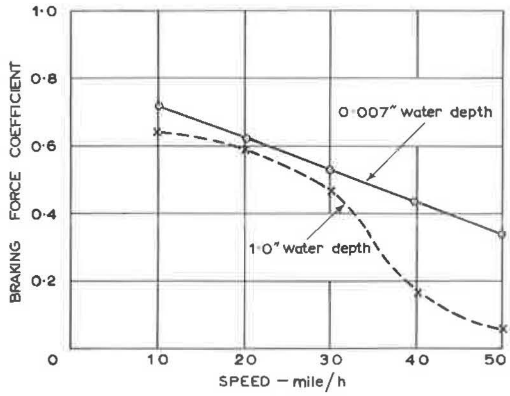

Figure 8. Effect of water depth on results of locked-wheel braking tests on a smooth surface with a patterned tire.

Table 2 shows that (a) in testing the slipperiness of roads there may be some disadvantages, as far as the consistency of the results is concerned, in attempting to carry out measurements at too high a speed unless very precise control over the depth of water in front of the test tire can be maintained, and (b) where front and rear wheels follow in the same tracks, rear wheels have the advantage of running over parts of the surface from which a great deal of water will have been cleared, or splashed away, by the front wheels. At the higher speeds this could clearly make some substantial increases in the adhesion available at the rear wheels, and it may be an important factor to consider when attempting to deduce behavior of four-wheeled vehicles at high speeds from coefficients measured with single- or twin-wheeled testing machines.

Of course, if the water is at all deep, vehicles running into it will experience a great deal of drag. Decelerations of the order of 0.5 g have been observed in a vehicle running into $2\frac{1}{2}$ in. of water at 45 mph (19). Under such conditions tire/road friction coefficients fall to low values. Test results illustrating this are shown in Figure 8.

HEAVY WHEEL LOAD SKIDDING MACHINE

Although the small trailer machine has been used quite extensively for various investigations, it has always been recognized that its wheel load (320 lb), tire size (4.00 x 8) and inflation pressure (20 psi) are such that it is difficult to infer from the results

Figure 9. Heavy wheel load skidding machine showing test wheel mounted on calibration rig.

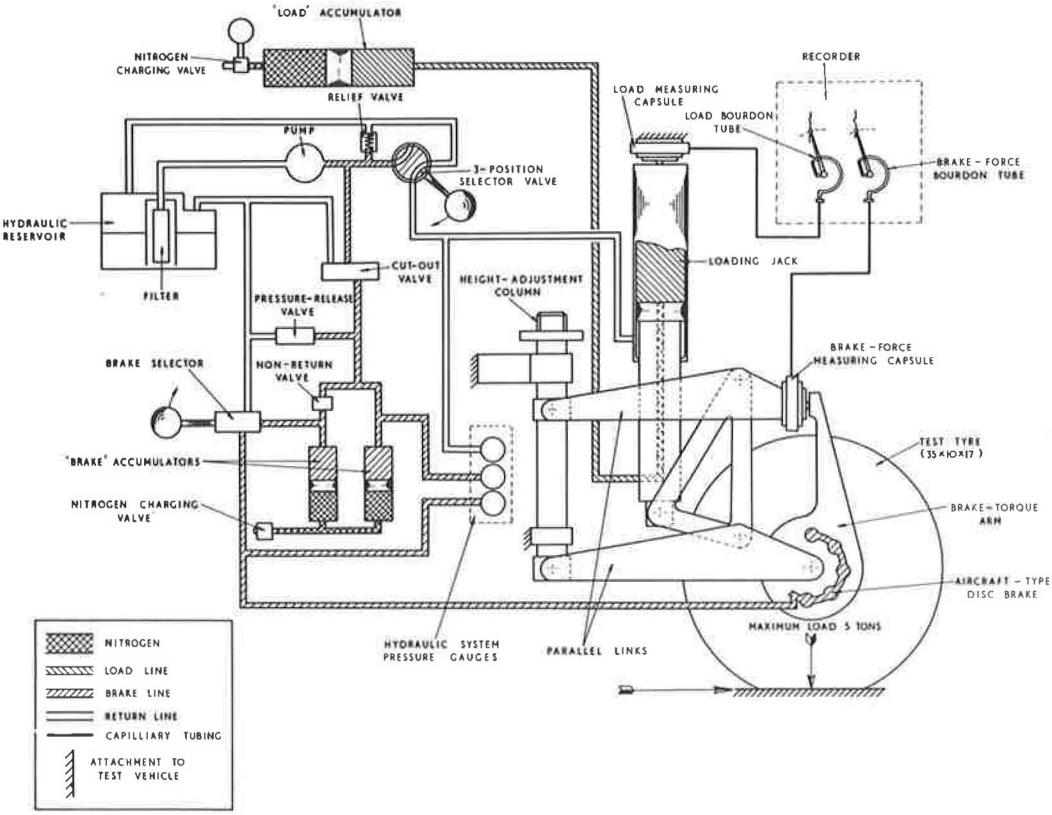

Figure 10. Heavy wheel load skidding test vehicle—diagrammatic arrangement of hydraulic systems and test wheel apparatus.

what values of coefficient are likely to apply under the same conditions for the wheels of cars and even more difficult for the wheels of aircraft.

In cooperation with the Ministry of Aviation, the Laboratory has, therefore, begun a new investigation into the effects of heavy wheel loads and high tire pressures on friction on wet surfaces. To carry out the tests a new testing machine was required and after considering various alternatives, a prototype of a fire-crash tender capable of speeds of 60 mph was specially modified for the work by the British Fighting Vehicle Research and Development Establishment. Some details of this vehicle showing the test wheel on its specially developed calibration rig are shown in Figure 9. The vehicle, which weighs 11 tons, has four-wheel drive and independent suspension on all wheels and is powered by a rear-mounted 240 B.H.P. engine. The vehicle can accelerate from rest to a test speed of 60 mph in 4,000 ft.

Details of the arrangement of test wheel and hydraulic recording system are shown in Figure 10. An aircraft test wheel and brake mounted in a parallel link suspension are used, and for considerations of stability when testing, the wheel is set within the wheelbase on the centerline of the vehicle and just behind the front wheels. By adjusting the nitrogen pressure in the loading accumulator the load on the test wheel (which is taken off the other wheels) can be set to any desired value up to a maximum of 5 tons. With this range of loadings, inflation pressures are over the range from 25 to 320 psi can be used.

On test the vehicle is first accelerated up to the desired speed and then, when it is running straight towards the test section, the test wheel is lowered. Enough load remains on the front wheels to provide sufficient traction, and to make small corrections to the steering the test wheel is arranged to pivot through a small angle about its

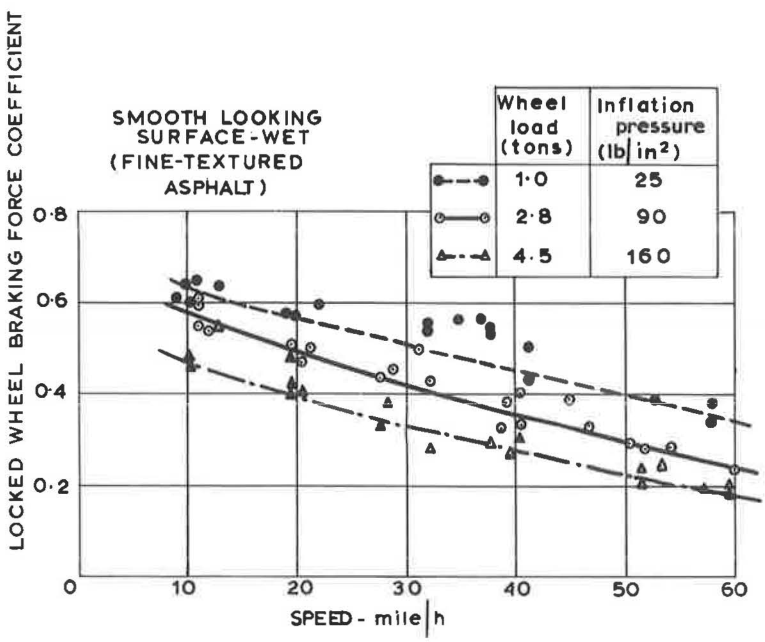

Figure 11. Relation between locked-wheel braking force coefficient and speed for various wheel loads and inflation pressure.

mounting column. On reaching the test section the wheel is braked, and immediately the ram is operated to raise the test wheel so that the vehicle can be braked to a standstill in the normal way.

Test results obtained with the machine are shown in Figures 11 and 12. In these experiments wheel loads and inflation pressures were adjusted to values shown to maintain an overall contact area of 72 sq in. for the tire in contact with a smooth surface. The results in Figure 11 show that there is a fall in the values of the locked-wheel coefficient when the higher values of the wheel load and inflation pressure are used. Thus it is very clear that under these conditions factors other than the pressure available to displace the water from the contact area are having an important influence on the coefficients obtained. Figure 12 shows that on slippery rough-coarse-textured surface results were rather different, and for speeds greater than 30 mph there appears to be little, if any, effect of load or inflation pressure on the coefficients obtained.

The results indicate that there are some fundamental differences in the mechanism of tire/road friction on these two main types of surface. This was further emphasized by the kinds of tire wear produced by locking the wheel on the different surfaces. As the load and tire pressure were increased there tended to be increasing amounts of tire wear when the wheel was locked, in spite of the fact that the tests were being made on wet surfaces. This increased wear may be a pointer to the mechanisms responsible for the decreased coefficients which were found. On the smooth or fine-textured surfaces fine particles of rubber were abraded from the surface of the tire while on the coarse-textured surfaces. The scalding type of wear associated with heating below the surface of the tire was produced.

These results were obtained in locked-wheel tests, but an aircraft type of "anti-skid" braking control system has now been installed on the vehicle. This has proved extremely effective in reducing tire wear and tests are continuing.

CONTROLLED SLIP SKIDDING MACHINE

Besides reducing tire wear under high/wheel loads anti-locking braking systems have the advantage of enabling a vehicle to use the better adhesion which can be obtained on

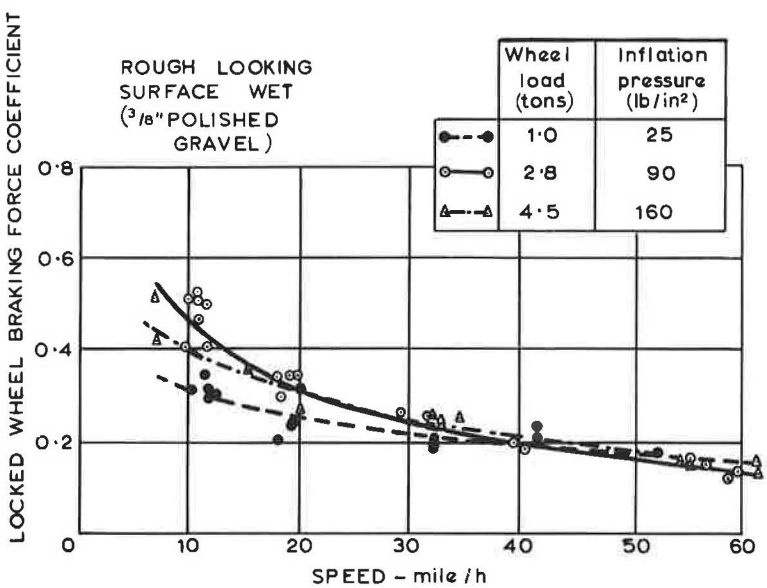

Figure 12. Relation between locked-wheel braking force coefficient and speed for various wheel loads and inflation pressures.

wet surfaces before the wheel is locked. To enable the braking force/slip relation to be studied more closely and also the effect of combined braking and cornering forces, a new machine has been built at the Laboratory (Fig. 13). The test wheel is installed within the wheelbase of a medium-sized truck, so that it runs just clear of the track of the nearside wheels. In developing the apparatus, use was made of a variable-ratio hydraulic transmission system. This comprises a variable displacement hydraulic motor coupled to a power take-off on the transmission of the towing vehicle, and this motor is connected to a similar but fixed displacement hydraulic pump driven through a suitable shaft and gearbox from the test wheel. This arrangement gives a "regenerative" braking system, and by controlling the displacement of the motor the test wheel can be made to turn with any desired rate of slip between free rotation and complete locking.

To facilitate testing, a direct-reading digital slip-meter has been developed. This consists essentially of two transistor counting units and some associated "gating" circuits. The first unit is used to count pulses from a photoelectric pulse generator coupled to one of the undriven wheels of the vehicle, and this unit controls the gating of the second unit. The second unit counts similar pulses from the test wheel. When both wheels are running freely they both generate pulses at the same rate. The gating of the second unit, however, is arranged to open or close at each successive count of 100 on the free-running wheel. If the rate of slip of the test wheel is now S percent, this second unit will only count 100-S pulses while the gate is open, and it is this count which is displayed to the operator on the digital indicator while the next count is being made. For all practical purposes a continuous indication of wheel slip is obtained, and it becomes a simple matter for the operator to set the slip control lever to give whatever rate of slip may be required.

To measure the wheel load, and forces in the plane of the wheel, and at right angles to it, a compact cruciform wire strain-gage unit has been fitted between the socket of the upper kingpin ball join, and the casing of the hub unit. Signals from this strain-gage unit are recorded on photographic film using a multi-channel galvanometer recorder which is sufficiently sensitive to record the signals direct, without amplification. Building the strain-gage unit into the hub casing in this way has an important advantage in that when the plane of the wheel is turned, the gages turn as well, and the forces both

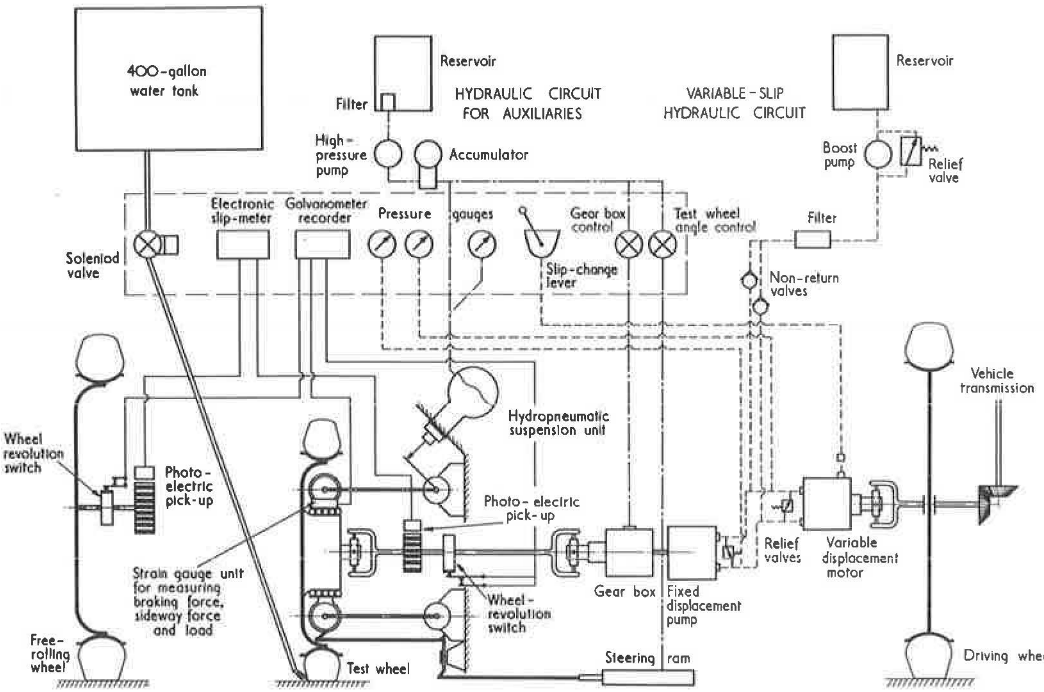

Figure 13. Apparatus for investigating skid-resisting properties under conditions of combined braking and cornering.

in, and at right angles to the plane of the wheel continue to be measured by the same arms of the strain-gage unit.

At present, tests with the machine have been chiefly concerned with following the coefficient/slip relationship in braking (with the wheel traveling straight ahead). Figure 14 shows preliminary results obtained under these conditions using a smooth tire, and a patterned tire, on two smooth or fine-textured surfaces, and on two coarse-textured surfaces. In most instances the coefficients obtained at 10-20 percent slip are higher than the locked-wheel values. However, the optimum slip, and more particularly, the precise form of the coefficient/slip relationship shows some marked differences. This can be seen, for example, in comparing the results given by the smooth tire, and the patterned tire, on the mastic asphalt surface. In the curves, in some instances, there was sharp drop in the coefficient values between 90 and 100 percent slip. It appears that on wet surfaces an anti-locking system could often be beneficial in improving braking even if it was not possible to modulate the brake pressure so as to keep the slip very close to the optimum value.

FRONT WHEEL BRAKING TESTS

Additional evidence of the importance of the difference between "peak" and "sliding" values of braking force coefficient especially at the higher speeds has been obtained in another type of skidding investigation carried out on the Laboratory's Research Track. In this investigation use was made of the front wheel braking technique first described by Lister and Kemp (20). Experiments were made to explore the possibility of studying the effects of tread pattern, tread composition, and surface texture at moderately high speeds by direct measurements of braking effects using an ordinary vehicle with the minimum of modification.

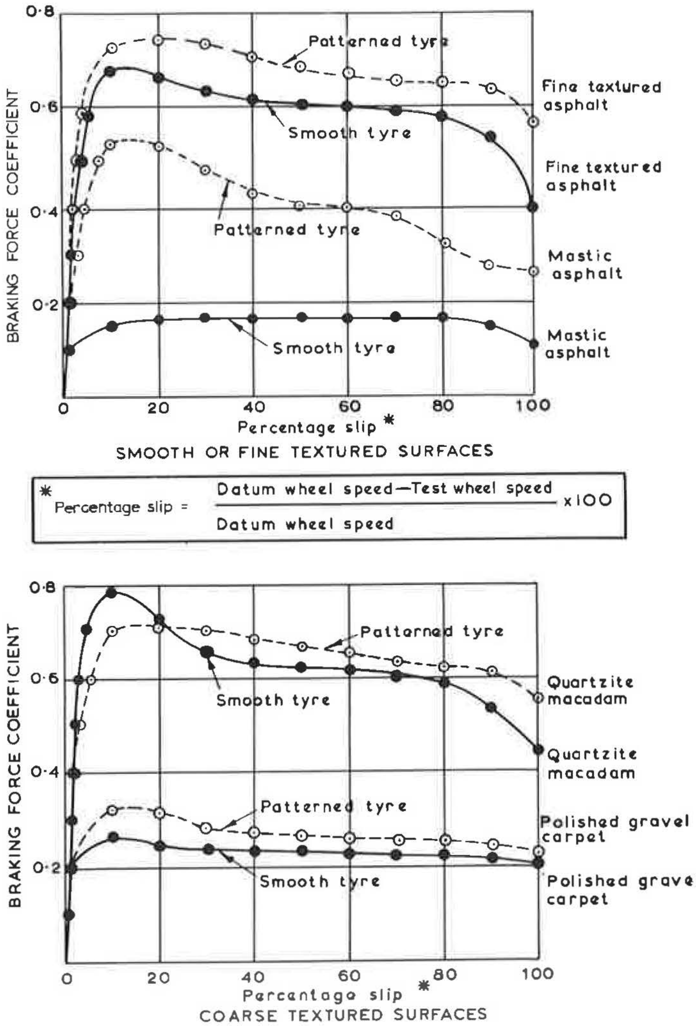

Figure 14. Braking force coefficient/slip relationships obtained at 30 mph with 500 X 16-in. tires on various wet surfaces.

In the past this has frequently been done by making measurements of the skidding distances of a vehicle with all its wheels locked from various speeds. Experience has shown that with this method difficulties can arise through the failure of the vehicle to stay on a reasonably straight course when the initial speed is of the order of 40 mph or more. The front wheel braking technique offers an effective way of overcoming these difficulties because a vehicle with only the front wheels braked tends to be directionally stable, and to continue on a straight path even after the front wheels have locked. An additional advantage (20) is that when the front brakes are applied slowly, the deceleration of the vehicle generally passes through a clearly defined maximum as the rate of slip of the wheels attains the value which gives the peak coefficient of friction between tire and road, before the wheels lock and slide. Each test, therefore, gives two values of braking force coefficient; the peak or maximum value corresponding with a comparatively low rate of sliding of the tire over the road and the sliding value obtained when the wheels are completely locked.

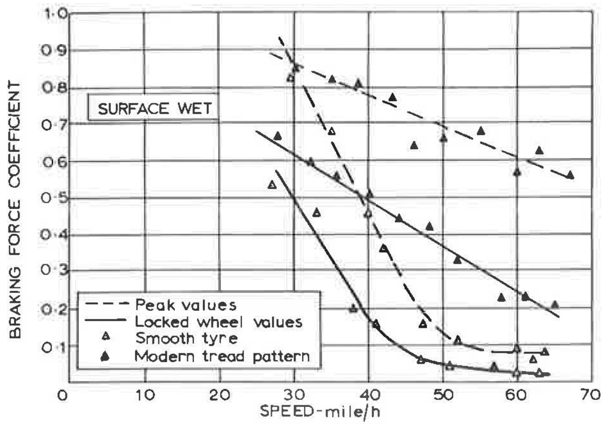

Figure 15. Results obtained in front wheel braking tests on a fine-textured surface (fine-textured asphalt).

To achieve this result the hydraulic braking system of the test car was modified by the addition of electrically operated valves so that, by operating a switch the rear wheel brakes could be disconnected and a needle control valve included in the front brake line. In this way the rate of application of the front brakes when full pressure was applied to them could be adjusted according to the skidding resistance of the surface. To measure the deceleration of the vehicle a potentiometer type of accelerometer was used, the output being recorded on one channel of a multi-channel recording galvanometer. Signals were also recorded from photoelectric pulse generators indicating the rotation of each of the wheels and from a wire strain-gage pressure transducer used to measure the hydraulic pressure being applied to the front brakes.

With these arrangements deceleration measurements were made on a number of test surfaces on the Laboratory's Research Track at speeds between 30 and 65 mph, when the surfaces were flooded with water to a depth of the order of 0.1 in. Knowing such factors as the weight distribution and height of the center of gravity, the deceleration values were then converted into the equivalent peak or locked-wheel braking force coefficients at the front wheels.

The set of curves in Figure 15 compares the results of tests on a smooth-looking surface with a good sandpaper texture and two sets of tires, both of the same rubber composition, but one set being quite smooth and the other having a typical modern tread pattern. It is evident that on a surface such as this, where speed affects the value of the coefficient, the tread pattern makes an important contribution to safety.

It is also evident from Figure 15 that the rate of slip is a most important factor particularly with the patterned tire, the peak coefficient at 60 mph being more than twice the value with locked wheels. With the smooth tires the braking force coefficients at this speed are very low, and even the peak coefficient is only of the order of 0.10. Thus on this one surface at a single speed, depending on the conditions, a wide range of coefficients is likely to be obtained.

Results obtained using the same two sets of tires on a rough coarse-textured surface are shown in Figure 16. By comparison with Figure 15 these results clearly show the superiority of rough coarse-textured surfaces in giving higher coefficients at the higher speeds. They also show the smaller influence of speed and the reduced importance of tread pattern effects on surfaces of this type. Also, on this surface it will be seen that the peak coefficients obtained are roughly twice as large as the sliding values with locked wheels. On surfaces covered by the tests it was found that peak braking force coefficients of the order of 0.5-0.8 were attainable under wet conditions at 60 mph where the corresponding locked-wheel values using some of the most modern tires available only lay between 0.20 and 0.25.

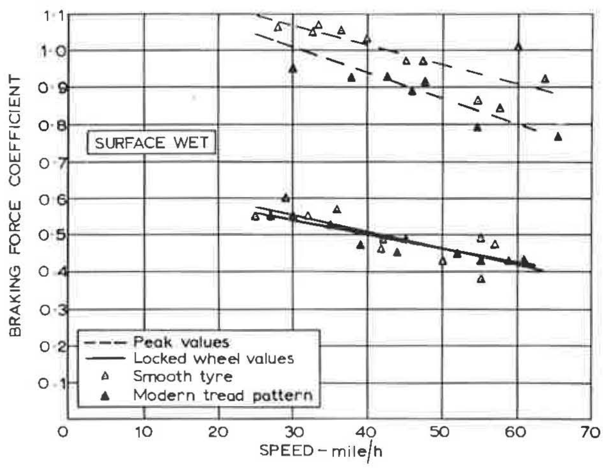

Figure 16. Results obtained in front wheel braking tests on a rough coarse-textured surface.

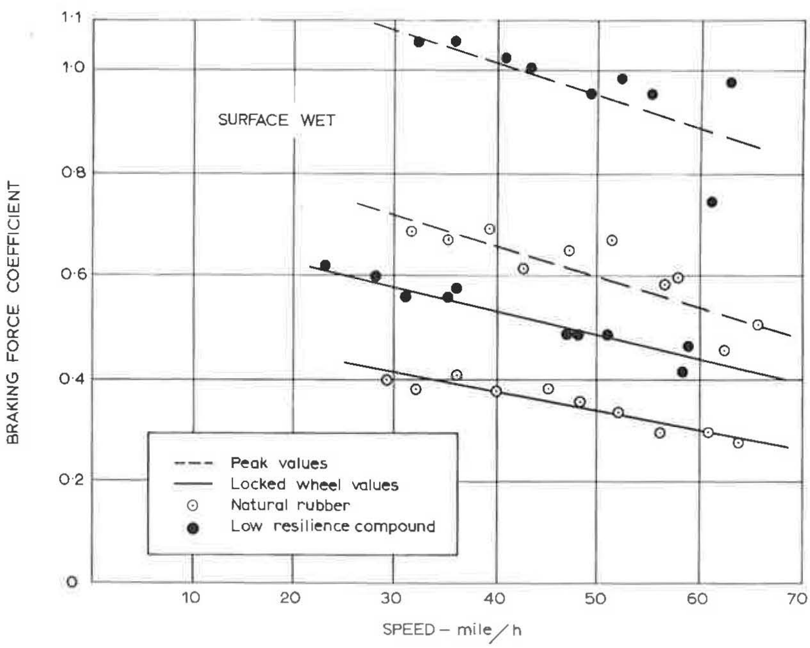

Figure 17. Effect of tread rubber resilience on a coarse-textured surface with smooth tires.

Other tests made during the experiments confirmed the gains obtained by using low resilience rubbers for the treads of tires. One such comparison is shown in Figure 17, where results are compared for two sets of smooth tires on the type of surface where tread patterns have little effect.

These results show that the front wheel braking technique, in conjunction with the facilities of the Crowthorne Research Track, promises to make an extremely direct and valuable means of studying tire/road friction problems over a range of speeds and test conditions which are less readily obtainable with conventional testing machines.

CONCLUSION

From the recent developments in testing techniques and facilities for work on skidding problems at the British Road Research Laboratory, the following conclusions are given:

1. Rough coarse-textured surfaces are likely to give better values of skidding resistance at high speeds under wet conditions than are surfaces of the smooth or fine-textured type. In certain conditions coefficients on the rough coarse-textured surfaces have even been observed to increase in value again as the speed is raised.
2. At higher speeds, effects due to the inertia of the water on a wet road becomes increasingly important and this point is likely to assume increasing importance in skid-testing when higher testing speeds are attempted.
3. Results obtained with the heavy wheel load machine show that on a number of surfaces increases in wheel load and inflation pressure lead to reductions in the values of coefficient obtained.
4. Previous findings on the importance of tire tread pattern, the resilience properties of the tread rubbers and the importance of their interaction with the texture of the road surface are confirmed over a wide range of conditions.
5. When wheels are braked on a wet surface the rate of slip has a very important influence on the value of coefficient which is obtained. The precise form of the coefficient/slip relation is influenced by the texture of the surface and the tire's pattern and resilience properties. It is clear that there may be quite large differences between the "peak" and "sliding" coefficients, particularly at the higher speeds.

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Principles of Subjective Rating Scale Construction

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This paper reviews the basic principles of subjective rating scale construction that have been developed in psychophysics and applied psychology. Particular emphasis is placed on the subjective measurement of pavement serviceability. The rationale underlying the AASHO Road Test pavement serviceability rating system is examined on the basis of these principles. Although some of the deficiencies of this serviceability measuring technique are illustrated, the full impact of these principles cannot be assessed until further experimental studies are undertaken.

ONE of the most significant developments resulting from the recently completed AASHO Road Test was the formulation and definition of the concepts of serviceability and failure of highway pavements, reported by Carey and Irick (1). Although the concept of pavement serviceability has been used more or less intuitively for many years to gage the success of pavement designs, the significant contribution of this study (1) was to demonstrate that serviceability was quantifiable. Furthermore it was shown that serviceability is a psychological quantity or experience, and not a physical measurement derived from pavement surface roughness.

The technique developed for measuring both serviceability and failure at the Road Test is based on a subjective estimate procedure. Although the manner in which human beings gage serviceability is necessarily an empirical problem, the known facts of psychophysics, however, set certain valuable guidelines.

It is well established that psychological experiences are measurable. However, all psychophysical quantities are subject to potential bias and distorting factors. The fact that an observer can be influenced in what he reports does not mean that his psychological impressions are not quantifiable, but merely that the task of measurement is difficult. An observer is sensitive not only to the physical stimuli he is trying to measure, but also to a large number of other factors that can distort his judgment to varying degrees.

In view of the susceptibility of human observers to external influences in communicating their psychological impressions, most psychophysical investigations seek to establish a measurement scale of the psychological experience and to relate this to a scale of measurement of the physical stimulus. Routine estimates of a particular psychological magnitude are then made from measurements of the physical correlate. The pavement serviceability rating system described by Carey and Irick (1) was developed within this type of framework. However, it is apparent that this subjective measurement procedure was developed without full cognizance of the basic principles of subjective rating scale construction. It is the purpose of this paper to review the basic principles of subjective rating scale construction, and to examine the validity of the rationale underlying the Road Test pavement serviceability rating procedure.

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MEASUREMENT OF SERVICEABILITY

Measurement, in general, is concerned with the rationale involved in the construction of a measuring scale, as well as the properties that can be attributed to measurements executed with a scale. The measurement of the majority of the properties of objects are expressed in the real number system. The real number system possesses certain fundamental properties of which the most important are order and additivity. The order of numbers is given by convention. Additivity refers to the fact that the operation of addition (used here in the completely general sense) gives results that are internally consistent. In other words, equal differences can be determined from the numbers, such as 7 - 5 = 4 - 2, as well as equal ratios, such as 8/4 = 6/3.

If it is possible to assign numbers to the properties of objects such that the properties of objects designated by the various numbers have the same characteristics as the number system (that is, if an isomorphism exists), then the number system may be used as a mathematical model of the properties of the object. It is, therefore, of great analytical advantage if this isomorphism between the properties of numbers and the properties of objects can be established. The principles and manipulations of mathematics applicable to the number system may then be used to manipulate the properties of the objects themselves.

Measurement exists in a variety of forms depending on the extent to which the properties of the number system are reflected in the scale of measurement. Measurement scales are classified into four basic types, and the classification proposed by Stevens (15), and given in Table 1, is generally accepted. Each of the four scale types given in Table 1 reduces the completely arbitrary element in the assignment of numbers to property magnitudes to a different degree. Stevens (17) suggests that a fundamental technique for evaluating scales of measurement is by using the concept of invariance of scale values under transformations of the scale. Table 1 contains a brief description of the empirical rule or operation invoked in the measurement operation, the transformations under which each scale type remains invariant, and an example of each scale type. Inasmuch as the arbitrary element in the assignment of numbers to properties is restricted to a different degree for each scale type, the characteristics of the numbers that are available for meaningful use as a model of object properties are likewise restricted.

The measurement problem with respect to pavement serviceability, therefore, resolves to one of first deciding the level of measurement required, and second, developing a procedure for scaling serviceability at this level of measurement. If the serviceability is to be used for establishing maintenance or resurfacing priorities for example, then all that would be required would be an ordinal scale of measurement; that is, an ordering of the pavement sections. However, if the measure is to be used to establish statistical relations between the serviceability measure and other factors, such as pavement strength and environment, then pavement serviceability must be measured on at least an interval scale. For example, consider a hypothetical pavement section that was rated at three periods throughout its life as possessing serviceability ratings of 4.0, 3.0 and 2.0. Unless the difference in serviceability between the first and second ratings is equal to the difference between the second and third ratings, it would be meaningless to attempt to relate these changes in serviceability to, say, differences in axle coverages.

TABLE 1

A CLASSIFICATION OF SCALES OF MEASUREMENT[a]

Scale	Basic Empirical Operations[b]	Allowable Transformations	Example
Nominal	Determination of equality	Any one to one substitution	"Numbering" of football players
Ordinal	Determination of greater or less	Any increasing monotonic function	Moh hardness scale of minerals
Interval	Determination of equality of intervals	Any linear transformation	Temperature (°F)
Ratio	Determination of equality of ratios	Any linear transformation retaining natural origin	Length, density, temp. (Kelvin)

[a]After Stevens (17).
[b]The basic operations needed to create a given scale are those listed down to and including the operation listed opposite the scale.

PSYCHOPHYSICAL MEASUREMENT

Psychophysics is concerned with the determination of quantitative relationships between physical stimuli and corresponding psychological or sensory events. The notions of a stimulus continuum, a response or sensory continuum, and a judgment continuum must be introduced to comprehend the principles of psychophysical measurement.

A stimulus or physical continuum refers to changes in some physical property such as the frequency of a sound wave, frequency of vibration, amplitude of vibration, or weight in pounds. Corresponding to these physical stimuli are certain sensory experiences or response continua such as pitch, perceived frequency of vibration, or subjective weight. It is not possible to measure directly quantities on the response continuum because these may only be estimated by observing an external verbal or symbolic response of an observer; that is, in the form of an externally communicated judgment by an observer. It is from these judgments that evidence concerning the response continuum must be derived. The introduction of a third continuum is, therefore, necessary for logical interpretation of response or sensory continua and their relationship to the corresponding physical continua. The exact manner in which human beings detect and respond to such physical stimuli as vibration, noise, etc., is not clearly understood. Goldman (5) and Hornick (10) have discussed the functions of some of the anatomical and biological systems that detect vibrations and motions, while Stevens (16) has investigated some of the factors concerned with the perception of noise.

Existing psychophysical theory, therefore, presupposes the existence of a judgment continuum paralleled by a response continuum, and through this relationship the judgment continuum is also related to the stimulus continuum. Common practice in psychophysics has been to assume a linear regression relating the judgment and response continua with perfect correlation. However, this correlation is not always perfect, and the nature of this correlation is discussed in more detail later in this paper.

Guilford (7) has provided an exhaustive review of the psychophysical scaling procedures that have been developed and used to establish relationships between these continua. Although they differ in detail, all psychophysical scaling methods may be considered as the combined effect on the sensory response of an observer, of an experimenter's operations of stimulation and instruction. Instruction refers to the response that is elicited from an observer, and the major difference in psychometric scaling methods is due to the nature of the response so obtained. Scaling methods are generally classified as judgment or response methods. With judgment methods the observer is instructed to assess the amount of a specified attribute possessed by a physical stimulus. With the response methods a specific attribute of the stimulus is not specified, and the observer is instructed only to indicate whether he agrees with or endorses a particular stimulus.

Rating scale methods are the most popular psychometric scaling procedures that depend on human judgment. Because of their widespread use, a significant body of principles governing their construction and use have been generated. They have been used in personnel evaluation, the reactions of individuals, aesthetic judgments, in the psychological evaluation of physical stimuli, etc.

PSYCHOLOGICAL MODEL OF RATING PROCEDURES

Several types of rating scales have been developed and widely used, but they are all essentially alike in that they require the assignment of objects by inspection, either along an unbroken continuum, or in ordered categories along the continuum. A numerical rating scale, an example being the Road Test serviceability scale, typically consists of a sequence of numbers defined by definitions or cues, and raters assign an appropriate number to each stimulus in line with these cues. Although other types of rating scales, such as graphic scales, have been developed and used, the numerical rating scale is the most appropriate scale type for pavement serviceability measurement if an interval scale of measurement is to be achieved.

On the basis of the previously outlined existing psychophysical principles which are discussed in detail by Guilford (7) and Torgerson (20), the following psychological model is proposed as the most appropriate model underlying the subjective determination of pavement serviceability.

1. Serviceability is a discriminable attribute of highway pavements and raters are

capable of making direct quantitative judgments of the amount of this attribute associated with any pavement section.

2. Each rater's judgment is considered to be a direct report of the level of serviceability of a pavement on a linear subjective continuum (interval scale) of this attribute. The origin and units in which the judgments are expressed may be arbitrary but they must remain constant.

3. Some variability in judgment with respect to the serviceability of any pavement may occur as is the case with any measurement procedure. This variability is treated as random error, and the individual estimates may be averaged to provide an estimate of the scale value of serviceability. It is implicitly assumed that the scale value estimate may be obtained from replications by an individual, or from judgments by a number of raters. That is, raters are assumed to be interchangeable.

Although not explicitly stated by Carey and Irick (1), this is essentially the psychological model assumed in the AASHO Road Test rating procedure. Guilford (7) however, has pointed out that several well known systematic errors occur in rating methods, and that these systematic errors must be removed from the raw judgments before psychological models of the aforementioned type may be considered to hold in actual rating studies. The most important of the recognized systematic rating errors occuring in ratings are, as follows:

1. The error of leniency which refers to the constant tendency of a rater to rate too high or too low for whatever reasons.

2. The halo effect which refers to the tendency of raters to force the rating of a

TABLE 2

SERVICEABILITY RATING MATRIX OF MINNESOTA AND INDIANA RIGID PAVEMENTS

Pvmt. No.	Serviceability Rating									Mean
	Rater 1	Rater 2	Rater 3	Rater 4	Rater 5	Rater 6	Rater 7	Rater 8	Rater 9	
201	1.8	1.0	2.4	2.0	0.7	1.2	0.7	1.0	0.9	1.3
202	1.9	1.1	1.7	2.4	2.7	2.1	1.5	1.2	1.4	1.8
203	2.4	1.8	3.1	2.7	1.7	2.4	1.5	1.7	1.5	2.1
204	4.4	4.1	4.5	4.0	3.8	3.8	3.6	4.2	4.1	4.1
205	4.4	3.9	4.5	3.5	3.5	3.8	3.1	3.8	4.0	3.8
206	3.6	2.4	3.7	2.6	3.2	3.2	2.9	2.9	2.4	3.0
207	3.5	2.4	4.2	2.4	3.0	3.6	2.5	3.2	2.6	3.0
208	3.4	2.4	4.1	2.2	2.7	3.2	2.8	2.8	2.3	2.9
209	2.9	1.8	3.8	2.4	2.9	2.3	2.2	2.5	2.3	2.6
210	1.9	1.2	2.0	1.9	1.6	1.6	0.8	3.0	1.5	1.7
211	4.9	4.7	4.8	4.2	4.7	4.3	4.1	4.3	4.2	4.5
212	4.9	4.2	5.0	3.8	4.5	3.8	4.3	4.3	4.3	4.3
213	4.3	3.4	4.2	3.8	3.8	3.6	3.1	3.7	3.5	3.7
214	4.2	2.9	3.2	3.5	3.1	3.8	3.3	4.0	4.0	3.5
215	4.6	3.4	4.8	3.3	4.6	4.3	3.7	4.5	4.0	4.1
216	4.5	3.2	4.7	3.6	4.4	4.0	3.0	4.3	3.5	3.9
217	1.7	1.0	1.2	1.8	1.5	1.6	0.7	0.8	1.0	1.3
218	1.8	1.0	1.6	1.8	0.9	1.6	0.8	0.7	1.0	1.2
219	3.6	1.8	3.8	2.9	2.7	3.2	2.9	3.7	2.0	3.0
220	4.8	4.2	5.0	4.4	4.6	4.1	3.9	4.4	4.3	4.4
401	4.0	3.8	4.5	4.3	3.8	3.2	4.1	3.9	4.0	4.0
402	3.9	3.2	4.8	3.4	3.7	3.7	4.1	4.0	3.8	3.8
403	3.7	3.3	4.7	3.6	3.9	3.4	3.5	3.5	2.5	3.6
404	3.3	2.1	4.2	3.7	2.9	3.0	3.6	3.2	2.5	3.2
405	3.0	1.8	3.5	2.5	2.6	2.3	3.1	2.2	2.2	2.6
406	3.0	2.5	3.2	3.1	2.8	2.7	3.1	2.7	2.4	2.8
407	2.8	1.6	1.8	1.5	2.4	2.0	2.5	1.5	1.0	1.9
408	2.6	1.5	1.8	0.8	2.8	2.4	2.5	2.0	1.0	1.8
409	3.1	1.9	2.7	1.5	2.2	1.8	3.0	2.1	1.0	2.1
410	3.0	2.0	2.2	1.8	2.7	2.2	3.1	2.1	1.3	2.3
411	2.5	1.5	1.7	0.8	2.3	1.5	2.3	2.0	1.3	1.8
412	4.0	1.8	3.1	2.3	2.5	2.7	3.1	3.0	2.3	2.8
413	4.1	4.3	4.9	4.1	4.6	3.4	4.3	4.5	4.0	4.2
414	4.1	4.7	4.7	4.4	4.4	3.5	4.1	4.6	4.1	4.3
415	4.0	4.6	4.9	4.3	4.5	3.4	4.7	4.2	4.2	4.3
416	2.1	0.6	0.4	0.4	2.0	1.7	0.6	1.0	1.0	1.1
417	3.0	1.7	3.0	1.7	2.2	1.8	2.6	2.5	1.0	2.2
418	4.4	4.3	4.9	4.3	4.3	4.4	3.9	4.6	4.0	4.3
419	3.4	2.0	3.6	2.6	2.9	3.4	3.1	2.5	1.4	2.8
420	3.2	2.3	3.0	2.2	3.0	2.9	2.7	2.3	2.0	2.6

particular attribute in the direction of the overall impression of the object rated.

3. The error of central tendency which refers to the fact that raters hesitate to give extreme judgments of stimuli and tend to displace individual ratings toward the mean of the group.

Table 2 contains the individual ratings of nine raters of the 40 Minnesota and Indiana rigid pavement sections surveyed in the rating studied reported by Carey and Irick (1). This rating matrix is examined in the following section for the presence of systematic errors of the aforementioned type, and the techniques that have been developed for removing these errors are outlined.

SYSTEMATIC RATING ERRORS

An analysis of variance of the rating matrix of Table 2 is given in Table 3. Both sources of variation, that of between raters and between pavements, are shown to be significant at the 1 percent level of significance. The previously described psychometric model of the rating procedure requires that raters be interchangeable, but the data in Table 3 illustrate that this requirement is violated in that the differences in ratings between raters are significant. Table 4 gives the mean rating and the standard deviation of ratings for each rater. This source of variation in ratings between ratings may be removed by transforming each rater's ratings to a distribution with mean and dispersion equal to the grand mean rating and the mean standard deviation.

The deviation of each rater's average rating for all pavement sections from the grand mean rating will indicate the magnitude of a rater's relative leniency error. The relative leniency errors, ΔR, are given in Table 4, and inspection of Table 2 reveals that, in general, this relative leniency error is constant for a given rater. The transformation of each rater's dispersion of ratings to a constant standard deviation is necessary because the contribution to the total variance of all ratings of a rater's ratings is proportional to the magnitude of the standard deviation of his ratings.

Guilford (7) has stated that a positive leniency error has been found to be the most common type of leniency error, but with the present data it is not possible to estimate the absolute leniency error. Guilford has further suggested that the descriptive cues may be adjusted to counteract this type of error by giving most of the scale range to degrees of favorable report. Evidently, raters anticipate a mean rating scale value somewhere near the cue good, or its equivalent, and a distribution symmetrical about that point.

The second type of systematic error common to ratings is known as the halo effect. The halo effect is considered to be a constant type error, and has been previously defined as the tendency of raters to force the rating of a particular trait in the direction of the overall impression of the object rated, and to that extent to make the ratings of some traits less valid. Symonds (18) suggests that the halo effect is more prevalent in ratings when a trait is not easily observable, or when the trait is not clearly defined. A relative halo effect would be manifested in significant interaction terms in an analysis of variance of a rating matrix in which the level of all attributes influencing a rater's ratings were systematically and quantitatively recorded. The available rating data cannot be analyzed for this error because the pavement traits were not systematically recorded.

The third type of error, known as the

TABLE 3

ANALYSIS OF VARIANCE OF SERVICEABILITY RATING MATRIX

Source	Sum of Squares	D.F.	Var.	F Ratio	P
Between raters	386	8	9.9	55	Significant
Between pavement	12	39	1.5	8.3	at 0.01
Remainder	57	312	0.18		level
Total	455	359			

TABLE 4

MEANS AND STANDARD DEVIATIONS OF RATINGS BY RATERS

Rater No.	Mean Rating	R	Std. Dev.
1	3.40	+ 0.44	0.87
2	2.58	- 0.38	1.20
3	3.50	+ 0.54	1.25
4	2.81	- 0.15	1.39
5	3.05	+ 0.09	1.07
6	2.92	- 0.04	0.98
7	2.89	- 0.07	1.09
8	2.98	+ 0.02	1.21
9	2.54	- 0.42	1.25
Mean	2.96		1.15

central tendency error, has been defined as the tendency of raters to judge stimuli in the direction of the average stimulus. One factor contributing to this error is that raters tend to displace ratings towards the mean of the group. Johnson (12) has explained this error from a statistical viewpoint, in terms of the regression towards the mean that always occurs when two variables are imperfectly correlated. In a judgment situation, this imperfect correlation results from imperfect discrimination of an attribute by an observer. The interpretation of this error, therefore, requires the introduction of the judgment continuum as distinct from the sensory continuum for logical explanation.

If a central tendency effect has occurred in a set of ratings, then the scale value estimates will have much less dispersion than the true scale values. The problem of removing this error resolves to one of establishing a relation between the true scale values, Tj, and the obtained scale values, Mj. The obtained scale values are the mean values of the individual estimates by raters. Guilford (7) has established this as a regression problem in which the obtained scale values, $\bar{M}j$, are predicted from the true scale values, Tj, and the dispersion of the single judgments, Aj, around these means represents the errors of prediction. Guilford assumes that the obtained and true scale values are perfectly correlated, and that the means of the two sets of values are equal. Further, it is assumed that the standard deviations of the true scale values and all single values are perfectly correlated. Guilford states that the justification of this assumption is that in the limiting case when the correlation between the values is perfect, Aj is perfectly predicted from Tj. Inasmuch as the correlation between Tj and Mj is perfect, all that is required is a linear transformation equation.

The standard deviation of all single ratings (transformed to equivalent distributions as previously described) is 1.18. The standard deviation of the obtained mean scale values is 1.04. Therefore, the transformation equation becomes

$$Tj = \frac{1.18}{1.04} (Mj - \bar{M}j) + \bar{M}t =$$

$$1.134\ Mj - 0.39 \qquad (1)$$

because $\bar{M}j = \bar{M}t = 2.94$.

Table 5 gives the relationship between the obtained scale values and the true scale values for the Indiana and Minnesota rigid pavements. The mean values of both sets of scale values (2.94), which are assumed to be equal, define the indifference point. The table shows that ratings below this point are overestimated, whereas ratings above this point tend to be underestimated. In general the greater the distance of a stimulus from the indifference point, the greater the error of estimation.

TABLE 5

COMPARISON OF OBTAINED AND TRUE RATING SCALE VALUES FOR INDIANA + MINNESOTA RIGID PAVEMENTS

Pavement Section	Scale Value True	Scale Value Obtained	Pavement Section	Scale Value True	Scale Value Obtained
201	1.3	0.9	401	4.0	4.0
202	1.8	1.6	402	3.8	4.0
203	2.1	1.9	403	3.6	3.6
204	4.1	4.2	404	3.2	3.2
205	3.8	3.9	405	2.6	2.1
206	3.0	3.1	406	2.8	3.0
207	3.0	3.1	407	1.9	1.8
208	2.9	2.9	408	1.8	1.7
209	2.6	2.4	409	2.1	2.1
210	1.7	1.4	410	2.3	2.2
211	4.5	4.7	411	1.8	1.6
212	4.3	4.6	412	2.8	2.8
213	3.7	3.8	413	4.2	4.4
214	3.5	3.7	414	4.3	4.4
215	4.1	4.4	415	4.3	4.4
216	3.9	4.1	416	1.1	0.9
217	1.3	1.0	417	2.2	2.2
218	1.2	1.0	418	4.3	4.5
219	3.0	2.9	419	2.8	2.6
220	4.4	4.6	420	2.6	2.6

The foregoing rationale assumed that the amount of under- or overestimation to be a linear function of the distance of the stimulus from the indifference point. Torgerson (20) has pointed out that a tendency exists for observers to force any series of stimuli into a normal distribution. If this were true, then scales constructed according to this rationale would not possess equal interval properties. However, the exact nature of the regression cannot be evaluated unless a corresponding physical continuum is available.

A second limitation of this rationale is concerned with the assumption that the discriminal dispersions at each scale value are equal. If these dispersions are not equal then the stimuli having greater dispersions may have regressed more toward the mean than stimuli having smaller dispersions. This would violate the assumption that perfect correlation exists between the true and obtained scale values. Inspection of the Road Test rating data suggests that the dispersions in ratings vary with the magnitude of the scale value; the dispersions at extreme scale values are less than the dispersions of the more central scale values. Insufficient data are available to properly evaluate this factor.

Newcomb (14) and Murray (13) have suggested that other significant errors, similar in nature to the halo effect, do frequently occur in subjective ratings. However, accepted methods for removing these errors from ratings have not been evolved due to an incomplete understanding of the precise nature of these errors.

Some of the quantifiable errors that frequently occur in ratings have been described and the following sections describe more general, but equally significant errors that distort ratings, and that must be minimized or removed from the ratings.

VALIDITY AND RELIABILITY OF RATINGS

The validity of ratings refers to the degree to which they are truly indicative of a psychological experience generated by a physical stimulus. The reliability of ratings refers to the consistency with which ratings are made, either by different raters, or by one rater at different times.

Rigorous validation of ratings is only possible through the comparison of ratings with more objective measures of the stimulus attribute. However, due to the complexity of many physical stimuli, precise measures of physical correlates are unknown. In such a case it is important to examine thoroughly the factors influencing the validity of ratings. Conditions may then be established which will be conducive to producing the highest possible validity in subjective estimates.

A most important factor affecting the validity of ratings is the definition of the attribute of an object that is to be rated. Guilford (7) has pointed out that many psychophysical investigations have demonstrated that an attribute name is primarily useful as a label, and used without adequate definition and without cues may become very misleading. Ghiselli and Brown (4) have pointed out that when personnel are rated on the basis of a general or overall trait there is greater probability of error, because different raters will base their judgments on different aspects of the performance included under the general trait name.

In view of the statements by Carey and Irick (1), Hveem (11), Housel (9), and Wilkins (21), there is some confusion concerning the exact nature of pavement serviceability. The terms pavement serviceability and pavement roughness have been used interchangeably. Pavement roughness refers to the distortion of a pavement surface from the geometry of the designed surface. The serviceability and failure of an engineering design can only be defined relative to the purpose for which a design has been provided. The purpose of a highway pavement is, as has frequently been stated, to provide a surface of adequate riding qualities throughout the life of a pavement. The riding quality afforded by a particular pavement section is a subjective experience and must be measured as such. The absolute riding quality is not a unique subjective characteristic but depends on the interrelationship of the pavement roughness, vehicle, and vehicle occupants. An absolute scale of riding quality would require the establishment of absolute levels of subjective experience that result from particular vibrational environments. A particular pavement section would, therefore, exhibit a wide range of riding qualities depending on the properties of the vehicular system using it. Consequently, pavement serviceability must be operationally defined in terms of the relative riding quality for each highway user. It is apparent that rating efforts at the Road Test were directed toward obtaining subjective estimates of the pavement distortion and deterioration, and not to obtaining estimates of the subjective experiences of riding quality.

Guilford (7) and Ghiselli and Brown (4) have provided further information of many of the other factors that are known to influence the validity of ratings.

The reliability of ratings is commonly defined operationally as the proportion of observed variance that is true variance. One technique for estimating the reliability of ratings is by re-rating a given set of physical stimuli and correlating the two sets of ratings. Guilford (7) has suggested that such a technique is susceptible to spurious correlation due to the memory of raters.

Ebel (1951) has described a method of estimating the reliability of ratings which is based on an analysis of variance of the ratings. The reliability of ratings for a single rater is given by

$$r = \frac{Vp - Ve}{Vp + (k - 1)\,Ve} \tag{2}$$

while the reliability of the mean ratings of the raters is given by

$$r = \frac{Vp - Ve}{Vp} \tag{3}$$

in which

r = reliability of ratings,
Vp = variance between pavements (or other stimulus),
Ve = variance of residuals, and
k = number of raters.

The reliability coefficient can be readily computed for any rating matrix. Although the coefficient is not particularly meaningful for a single matrix, it is invaluable in the evaluation of various scale formats as is pointed out later in the paper.

SCALE CONSTRUCTION AND FORMAT

To assist raters in arriving at quantitative judgments at an interval scale level of measurement, the attribute definition should be supplemented and reinforced by cues or descriptive phrases. Champney (2) after an extensive study has listed criteria that may be used as a guide to the systematic development of cues for rating studies. The most important of these recommendations are that cues should apply to a very short and particular range on the continuum to provide raters with definite anchors, and that the cues for each trait should be unique to that trait. In particular, cues of a very general character such as "excellent," "poor," etc., should be avoided. The determination of the optimum scale format for a particular rating situation is necessarily an empirical problem. The error of leniency and the central tendency effect may be minimized by judicious selection of cues. It was previously pointed out that a positive leniency error may be minimized by using only unfavorable cues, because raters anticipate a mean rating somewhere near the cue "good" or its equivalent. The error of central tendency may be counteracted by adjusting the strength of the descriptive phrases. Greater differences in meaning may be introduced between steps near the extremities of the scale than between steps near the central area.

A most important parallel problem concerns the number of steps or categories that should be employed in a rating scale. The Road Test scale uses five categories. Wilkins (21) has pointed out that a ten-category scale is used in the Canadian rating studies. If the steps in a rating scale are too coarse, the raters' powers of discrimination cannot be effectively used. However, loss of reliability may result from steps that are finer than the raters' discrimination abilities. Symonds (19) has suggested that optimum reliability in ratings will be obtained by seven categories. Other studies have shown that the optimum number of steps varies considerably with the nature and complexity of the trait being rated. Consequently, the optimum number of categories may only be determined by experimental evaluation, and one objective criterion for optimizing the number of categories is the reliability coefficient, defined in Eqs. 2 and 3.

INTERVAL SCALE PROPERTIES

The basic psychometric model previously described has proceeded under the assumption that raters are capable of judging stimuli on an equal interval scale. Although the previous sections have been devoted to describing various errors and distortions common to ratings, no explicit provision for testing this fundamental assumption was proposed. Torgerson (20) has pointed out that consistency of judgments, or reliability, is not an adequate criterion to evaluate this assumption. For example, a criterion of judgment consistency cannot distinguish between equal interval judgments and judgments of ordinal positions of stimuli. Both Torgerson (20) and Stevens (16) suggest that the best approach to this evaluation lies in an examination of the invariance characteristics that an interval scale must possess.

It was pointed out in Table 1 that an equal interval scale is one in which the numbers assigned to stimulus magnitudes must be determined within a linear transformation of the form $y = a + bx$. That is, ratios of differences between scale values must remain invariant upon transformation. Thus, for a subjective estimate scale without a physical correlate, the minimum requirement for an interval scale would be that the ratios of differences in scale values assigned to at least three stimuli should remain invariant when the stimuli are scaled under differing experimental conditions. That is, a linear relation should exist between the different sets of scale values. Such an evaluation is not feasible with the available Road Test rating matrices.

Guilford (7) has concluded from the limited number of studies carried out to evaluate the measurement status of rating scales, that they may be regarded as having the status of ordinal measurements and only approach the status of interval measurements. He further suggests by the various methods of scaling and correction that they can be more or less successfully transformed to interval scale measurements.

One factor important to achieving valid and reliable ratings is the notion of scale anchoring. The anchoring concept refers to those conditions that control the origin and unit of the subjective continuum in which raters will report their judgments of magnitude. Experimental studies of anchoring effects have demonstrated that the unit and origin in which judgments are expressed are not absolute, but are functions of a particular experimental situation. Raters adjust the origin and unit to the distribution of the particular set of stimuli rated and to the rating categories allowed. The majority of psychophysical investigations have achieved invariant units and origin by systematic training of raters.

The preliminary rating studies at the Road Test were carried out to establish a common unit and origin for the rating panel. However, the application of a subjective serviceability rating procedure to pavement rating on a national scale may result in significant discrepancies in ratings from area to area. Rating panels from regions in which a wide distribution of pavement serviceabilities exists might be expected to establish a different subjective unit of serviceability than panels from regions possessing a much narrower range of pavement serviceabilities. Similarly, the origin of ratings would be expected to be a function of the average serviceability level existing in a region.

PHYSICAL CORRELATE OF SERVICEABILITY

An immediate problem in the serviceability measurements of pavements is to establish a suitable and rational physical correlate of pavement serviceability. Measurement of such a correlate, along with the subjective measurements, would then allow pavement serviceability to be scaled by more rigorous techniques than the methods used for purely subjective estimates. These scaling methods are well described by Torgerson (20) and Guilford (7). Furthermore, available psychophysical evidence suggests that it is unreasonable to apply a subjective rating procedure to the routine measurement of pavement serviceability over a wide area.

It has been recognized for many years that the longitudinal distortion of pavement surfaces is a determining factor with respect to their riding qualities. The empirical relations developed at the Road Test between the subjective magnitudes of serviceability and certain physical measurements of the pavement surface have also demonstrated this point. The measurement of the variance of the pavement slope seems to be a most valuable measurement in this regard. Only limited success has been achieved in characterizing roughness profiles with the aid of such instruments as the BPR roughometer, and the Michigan profilometer. The observed randomness of pavement roughness indicates that some form of statistical characterization is required. Limited applications of spectral density techniques reported by Grimes (6) and Coleman and Hall (3) have met with some success. Further, spectral density functions of road roughness profiles can be fundamentally related to the vibrational environment produced in vehicles excited by the various pavements.

SUMMARY

1. Pavement serviceability is a subjective or psychological phenomenon and must be measured as such. It must be measured on a scale possessing at least interval scale status if the serviceability measures are to be used for statistical correlation with other pavement properties.
2. Existing psychophysical theory presupposes the existence of a judgment, a sensory, and a stimulus continuum. Knowledge of the sensory or psychological continuum can only be achieved by measuring judgments by observers. It is these judgments that are subject to bias and distortion by a variety of environmental factors, thus tending to invalidate the measurements of the psychological experiences.
3. The basic psychometric model underlying subjective rating procedures assumes that raters are capable of making direct quantitative judgments on a linear or interval scale of measurement. Several well known systematic errors are known to distort subjective ratings. These include the leniency error, the central tendency effect, and the halo effect. These errors must be removed from ratings before the basic psychometric model is valid.
4. An important factor influencing the validity of ratings is how well the attribute to be rated is defined. The Road Test definition is very general in nature and appears to have resulted in some confusion with respect to just what attribute is being rated. Pavement serviceability must be defined as the relative riding quality afforded each highway user.
5. A useful objective measure of reliability (reproducibility) of ratings is the reliability coefficient which is defined as the proportion of observed variance that is true variance.
6. A most important influence on the ability of raters to achieve interval scale status is the nature of the cues or descriptive phrases that are used to reinforce the definition of the subjective continuum. In addition to providing anchors, judicious arrangement of the cues may be used to counteract a positive leniency error and the central tendency effect.
7. An important factor concerning the scale format is the number of rating categories used. If insufficient categories are used, then the raters' powers of discrimination cannot be fully utilized. Loss of reliability may result from too many categories. A useful objective criterion of optimization of the number of categories, as well as the cue format, is the reliability coefficient.
8. No explicit provision for testing the ability of raters to achieve interval scale status is contained in subjective rating procedures. Consistency of judgment, or reliability, is an inadequate criterion. The invariance characteristics of scales may be used to test this assumption in the absence of known and measurable physical correlates.
9. Anchoring refers to those conditions that control the origin and unit of the subjective continuum in which raters will report their judgments of magnitude. Experimental studies of this phenomenon have revealed that the unit and origin are not constant but functions of a particular experimental situation. These observations have important implications with respect to subjective serviceability rating at a national level.
10. It is generally considered that subjective rating procedures achieve the measurement status of ordinal scales, and only approach the status of interval scales. The

various methods of scaling and correction allow transformation to interval scale measurements.

11. Although some of the more common distortions and biases to which ratings are vulnerable have been shown to be present in the Road Test ratings, a complete evaluation of the ratings is impossible. The need exists for the design of suitable experiments to evaluate some of the factors concerning scale format, anchoring, etc., that have been described.

12. A more rigorous scale of pavement serviceability cannot be established until a suitable physical correlate of pavement serviceability is established.

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A Modification of the AASHO Road Test Serviceability Index Formula

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•USE of the CHLOE profilometer in Texas indicated that this device for measuring road roughness tends to rank pavements having a coarse-textured surface too low on the serviceability scale. To offset this tendency, a hand-operated device for measuring coarseness of texture was developed, and a term for textural roughness was added to the AASHO Road Test formula for the serviceability index. The coefficient of the new term was evaluated by analysis of the subjective ratings given 43 flexible pavements by a 12-man panel of Texas highway engineers.

The new formula for serviceability index predicted the adjusted ratings with satisfactory accuracy and will be used on this project for calculating the serviceability index for flexible pavements. It is anticipated that minor modifications will be required in the texturemeter, its use and the subsequent PSI equations as additional rating data are analyzed.

THE SERVICEABILITY INDEX FORMULA

The objectives of Research Project 2-8-62-32, "Application of the AASHO Road Test Results to Texas Conditions," require the calculation of the serviceability index of each flexible pavement test section from the following formula developed at the AASHO Road Test (2).

$$p = 5.03 - 1.91 \log_{10} (1 + \overline{SV}) - 0.01 \sqrt{C + P} - 1.38 \overline{RD}^2 \qquad (1)$$

in which

p = the present serviceability index;

$\overline{SV}$ = the mean of the slope variance in the two wheelpaths, multiplied by 10^6;

$C + P$ = a measure of cracking and patching in the pavement surface; and

$\overline{RD}$ = a measure of rutting in the wheelpaths.

The serviceability index, p, is an estimate of the mean subjective rating which would be given the pavement by a cross-section of highway users, and represents the instantaneous ability of the pavement to serve high-speed, mixed traffic at the time it is rated (1).

The index (or the rating) is restricted to values from 0 to 5.0. The scale is divided into five categories (2) (Table 1).

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TABLE 1

Rating or Index	Description of Pavement
0 - 1	Very poor
1 - 2	Poor
2 - 3	Fair
3 - 4	Good
4 - 5	Very good

EFFECT OF SURFACE TEXTURE ON SLOPE VARIANCE

In Eq. 1 the term having the greatest effect on the serviceability index is the slope variance term, log $(1 + \overline{SV})$. Slope variance at the Road Test was measured by a specially developed instrument known as the AASHO Road Test profilometer. On the present project, slope variance is being measured by the CHLOE profilometer (3) an instrument also developed at the Road Test but not regularly used there (Figs. 1 and 2).

Use of the CHLOE profilometer on Texas pavements has shown that the slope variance arising from roughness of surface texture cannot be distinguished from that resulting from objectionable undulations in the pavement. (The Road Test profilometer was equipped with an electronic component which filtered out high frequency voltage fluctuations arising from roughness of surface texture.) As a result, the serviceability indexes computed from CHLOE profilometer readings are generally too low (roughness too high) for rough-textured pavements that otherwise exhibit no objectionable roughness. Chastain and Crawford (4) report the same problem encountered on a study of the CHLOE and several roughometers in South Dakota in 1962. Informal discussions indicate similar conditions from various other sources.

METHODS USED IN MODIFYING THE SERVICEABILITY INDEX FORMULA

There being no effective method known to the project staff for damping out the high frequency vibrations of the CHLOE profilometer slope wheel mechanism arising from rough-textured surfaces, it was decided to develop a hand-operated device for measuring roughness of surface texture, and to add to the Road Test serviceability formula a term for surface texture that would serve to correct the serviceability index when necessary.

Figure 1. CHLOE profilometer in use. Man behind profilometer records cracking, patching and rut depth, to be used with CHLOE and texturemeter data in calculating serviceability index. Profilometer is loaded in towing vehicle for travel between test sections.

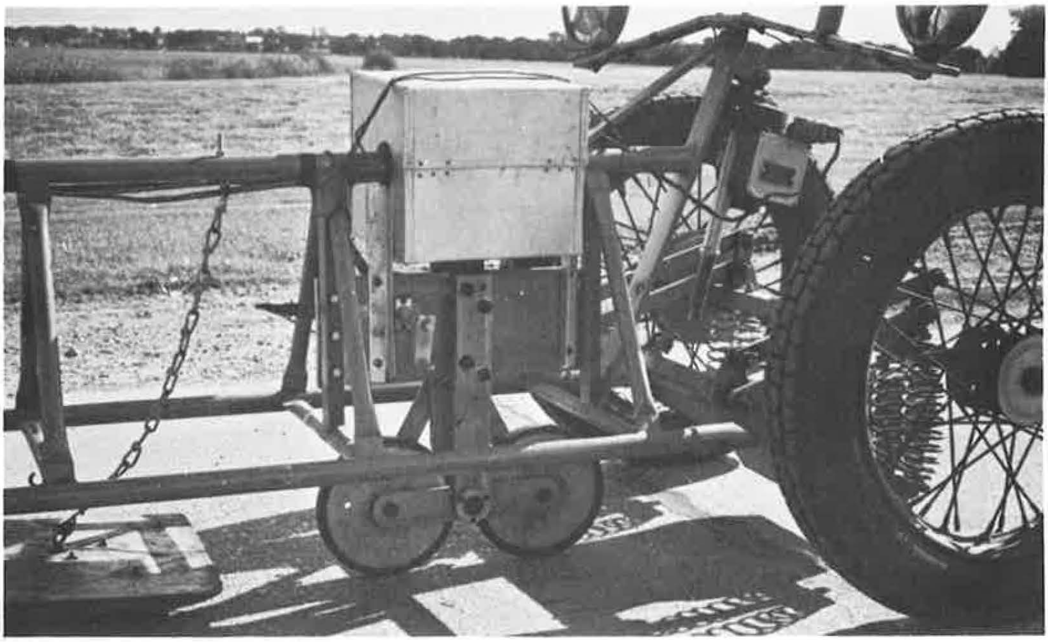

Figure 2. Close-up of slope wheels of CHLOE profilometer. Device measures angle between trailer tongue and link between slope wheels.

The instrument developed for measuring coarseness of texture is known as a texturemeter (Figs. 3 and 4), consisting essentially of a series of evenly spaced, parallel rods mounted in a frame. The rods can be moved longitudinally, independently of one another, against spring pressure. At either end of the series of movable rods is a fixed rod rigidly attached to the frame.

Each movable rod is pierced by a hole through which passes a taut cord (fiberglass-reinforced nylon), one end of which is fixed to the frame and the other to the spring-loaded stem of a 0.001-in. dial gage mounted on the frame. When the instrument is in use, the rods are held in a vertical position with their ends resting against the pavement surface. If the surface is smooth, the string will form a straight line and the dial will read zero. Any irregularities in the surface will cause the cord to form a zigzag line and will result in a reading on the dial. The coarser the texture of the pavement, the larger will be the dial reading.

The readings given by an instrument of this kind are affected by the spacing of the rods and the distance between the fixed supports. In the texturemeter now being used, the rods are spaced at $^5/_8$ in., and the instrument spans a distance of 10 in. between fixed supports. Some consideration was given to constructing the device with random spaced rods and there is probably some merit to this idea. It was generally decided, however, that the costs and trouble of such construction was not merited because the placement of the stones in the surfacing is itself a random factor. As the number of readings increases, the probability of such problems occurring continues to decrease.

It was postulated that the serviceability rating, R, of a test section could be estimated from the following mathematical model involving textural roughness:

$$R = p + A_0 + A_1 \log(1 + T) \qquad (2)$$

in which p is given by Eq. 1, T is the mean reading of the texturemeter in the two wheelpaths in thousandths of an inch, and A_0 and A_1 are constants to be determined by analysis of the subjective ratings given a number of selected test sections by a rating panel.

SELECTION AND RATING OF TEST SECTIONS

To provide ratings and other data required by the aforementioned analysis, forty-three 2,400-ft flexible pavement test sections in District 9 near Waco, for which CHLOE profilometer data were already available, were selected and rated by a 12-man panel in December 1962, and January 1963. Texture measurements on each section

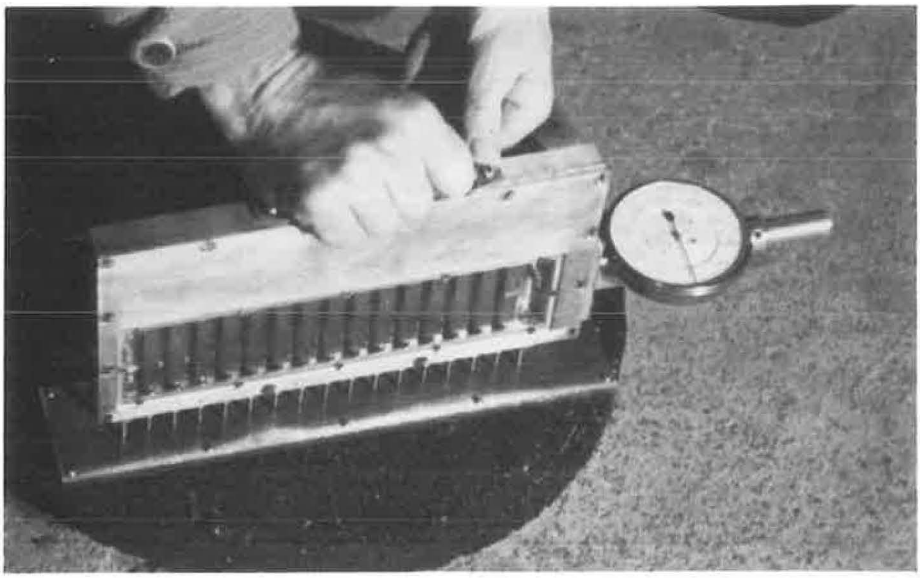

Figure 3. Texturemeter applied to flat, metal surface for zero reading.

Figure 4. Texturemeter applied to laboratory specimen of asphaltic concrete. Road surfaces give dial readings ranging from 0 to about 0.100 in.

were made in January 1963. Mean ratings, R, and texturemeter readings, T, are summarized in Table 2.

Each value of T in the table is the average of 40 readings by the texturemeter, 20 in each 2,400-ft wheelpath.

The rating R, of a section was calculated as follows:

An average of the 12 individual ratings was calculated. If one or more individual ratings deviated from the average by 0.8 or more, those ratings were eliminated, and the average of the remaining ratings was taken as the rating for the section. Under this procedure, the number of ratings averaged to obtain the mean for a section varied from 9 to 12, the most frequently occurring number being 11.

TABLE 2

Test Section	R	p	R - p	T	R′	p′	R′ - p′	p_c	Error ($R' - p_c$)
(a) Asphaltic Concrete Surface									
15- 6-1	4.1	4.4	- 0.3	2.4	4.5	4.2	0.3	4.4	0.1
15- 7-1	4.0	4.2	- 0.2	4.0	4.4	4.1	0.3	4.5	- 0.1
15- 7-2	3.8	3.9	- 0.1	6.9	4.2	3.8	0.4	4.3	- 0.1
15-14-1	3.9	4.0	- 0.1	2.8	4.3	3.9	0.4	4.2	0.1
15-14-2	2.7	3.2	- 0.5	1.4	3.0	3.2	- 0.2	3.3	- 0.3
15-14-3	3.6	4.6	- 1.0	1.1	4.0	4.4	- 0.4	4.5	- 0.5
231- 3-1	3.9	4.3	- 0.4	0.8	4.3	4.2	0.1	4.2	0.1
231- 4-1	4.0	4.4	- 0.4	1.0	4.4	4.2	0.2	4.3	0.1
251- 1-1	3.3	3.5	- 0.2	4.2	3.6	3.4	0.2	3.8	- 0.2
251- 2-1	4.0	4.3	- 0.3	2.0	4.4	4.1	0.3	4.3	0.1
251- 3-1	4.0	4.2	- 0.2	3.9	4.4	4.0	0.4	4.4	0.0
258- 7-1	3.8	4.4	- 0.6	0.3	4.2	4.2	0.0	4.1	0.1
258- 7-2	3.6	4.0	- 0.4	0.0	4.0	3.9	0.1	3.7	0.3
258- 7-3	3.2	3.8	- 0.6	0.1	3.5	3.6	- 0.1	3.5	0.0
382- 2-1	3.8	3.6	0.2	2.2	4.2	3.5	0.7	3.7	0.5
183- 3-1	4.1	4.8	- 0.7	3.8	4.5	4.4	0.1	4.8	- 0.3
184- 3-1	4.0	4.6	- 0.6	1.0	4.4	4.4	0.0	4.5	- 0.1
209- 7-1	3.9	4.4	- 0.5	1.9	4.3	4.3	0.0	4.5	- 0.2
413- 2-1	4.0	4.5	- 0.5	1.1	4.4	4.3	0.1	4.4	0.0
833- 4-1	4.0	4.6	- 0.6	1.8	4.4	4.4	0.0	4.6	- 0.2
833- 4-2	3.8	4.4	- 0.6	1.8	4.2	4.2	0.0	4.4	- 0.2
209- 3-1[1]	3.5	4.6	- 1.1	3.2	3.9	4.4	- 0.5	4.7	- 0.8
209- 3-2[1]	3.6	4.6	- 1.0	3.2	4.0	4.4	- 0.4	4.7	- 0.7
(b) Surface Treatments									
55- 2-1	3.5	2.3	1.2	31.9	3.9	2.3	1.6	3.3	0.6
209- 2-1	2.6	2.2	0.4	38.6	2.9	2.2	0.7	3.3	- 0.4
386- 3-1	2.7	2.2	0.5	46.5	3.0	2.2	0.8	3.4	- 0.4
386- 3-2	3.2	2.2	1.0	54.3	3.5	2.2	1.3	3.4	0.1
386- 4-1	2.6	2.1	0.5	25.0	2.9	2.1	0.8	3.1	- 0.2
386- 4-2	2.6	2.1	0.5	12.0	2.9	2.1	0.8	2.8	0.1
398- 5-1	2.7	2.6	0.1	11.6	3.0	2.5	0.5	3.2	- 0.2
519- 3-1	3.2	2.4	0.8	27.7	3.5	2.4	1.1	3.4	0.1
590- 2-1	3.1	2.6	0.5	10.9	3.4	2.6	0.8	3.3	0.1
836- 2-1	3.0	3.3	- 0.3	2.1	3.3	3.2	0.1	3.4	- 0.1
836- 2-2	3.0	2.8	0.2	1.6	3.3	2.8	0.5	3.0	0.3
1665- 1-1	2.5	1.8	0.7	43.2	2.8	1.8	1.0	3.0	- 0.2
2305- 1-1	2.3	2.0	0.3	19.1	2.5	2.0	0.5	2.9	- 0.4
2395- 1-1	2.6	2.3	0.3	5.0	2.9	2.3	0.6	2.7	0.2
656- 1-1	3.6	2.4	1.2	40.0	4.0	2.4	1.6	3.5	0.5
833- 7-1	3.4	2.5	0.9	19.2	3.7	2.4	1.3	3.3	0.4
1078- 2-1	3.5	2.8	0.7	14.5	3.8	2.8	1.0	3.6	0.2
2625- 1-1	3.5	2.8	0.7	25.6	3.8	2.8	1.0	3.8	0.0
1054- 4-1[1]	2.4	2.6	- 0.2	29.4	2.6	2.6	0.0	3.6	- 1.0
1594- 2-1[1]	2.5	2.8	- 0.3	24.1	2.8	2.8	0.0	3.8	- 1.0

[1]Data from this section not used in analysis.

ANALYSIS

For use in a regression analysis, Eq. 2 was rearranged

$$R - p = A_0 + A_1 \log(1 + T) \tag{3}$$

Values of (R - p) were plotted as ordinates, and log (1 + T) as abscissa in Figure 5. Also shown in the figure is a plot of Eq. 4 obtained from the regression analysis

$$R - p = -0.77 + \log(1 + T) \tag{4}$$

The squared correlation coefficient was 0.78 and the standard deviation was 0.28.

Data for sections 209-3-1, 209-3-2, 1054-4-1, and 1594-2-1 were eliminated from this and the succeeding analysis as extreme values, because of the fact that the R - p values for these sections deviated from the values predicted by Eq. 4 by an amount considerably in excess of two standard deviations. The reason for the unusually low ratings—or unusually high serviceability indexes—for these sections was not apparent. As time permits, further study of these sections will be made.

ADJUSTMENT OF DATA

Eq. 4 shows that when T = 0, as for a very smooth-textured pavement, the serviceability index, p, exceeds the rating, R, by nearly 0.8 of a rating unit. Thus, in those instances where texture had no effect on the profilometer, either the subjective ratings were too low, or the serviceability indexes computed from the profilometer were too high, or both of these conditions existed.

It was obvious that most members of the rating panel were hesitant to rate any pavement in the "very good" category, as evidenced by the maximum panel mean of 4.1. It was the opinion of the authors (both former AASHO Road Test staff members) that

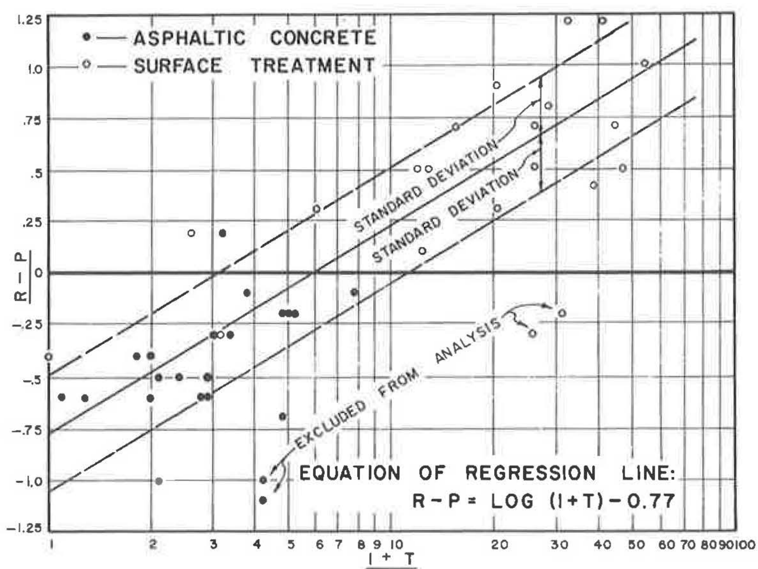

Figure 5. Difference between rating and serviceability index as a function of roughness of surface texture. Data are from Table 2.

the Road Test panel would have rated several of the District 9 pavements in the neighborhood of 4.5. To establish agreement between the national and the local panel, it was concluded that all local ratings should be adjusted upward by 10 percent, so that the maximum mean rating of 4.1 would be increased to 4.5, and other ratings would be increased proportionately. The column headed R′ in Table 2 gives the adjusted values of the rating.

On the other hand, examination of the profilometer data showed that this instrument had in several instances yielded serviceability indexes for subsections (1,200-ft half-sections) as high as 4.9 or 5.0. It was felt that these were too high, and that until the instrument could again be correlated with the Road Test profilometer, the serviceability indexes calculated from CHLOE data should be adjusted downward.

The original study correlating the CHLOE with the Road Test profilometer showed that a constant, 3×10^6, should be subtracted from the CHLOE slope variance in order to make its output agree with that of the Road Test instrument. This correction was used in calculating the values of serviceability, p, given in Table 2.

To achieve the desired reduction in serviceability index, the values of p′ given in Table 2 were calculated using a correction constant of 2.5×10^6. Values of p′ are somewhat less (one to two tenths, usually) than the corresponding values of p, when p is above 3.0. Below 3.0, the effect of the change in the correction constant is practically negligible.

ANALYSIS OF ADJUSTED DATA

A plot of R′ - p′ versus log (1 + T) is shown in Figure 6. The best fitting line through the data is

$$R' - p' = 0.18 + 0.81 \log (1 + T) \qquad (5)$$

The squared correlation coefficient was 0.70 and the standard deviation was 0.28. Eq. 5 may be written

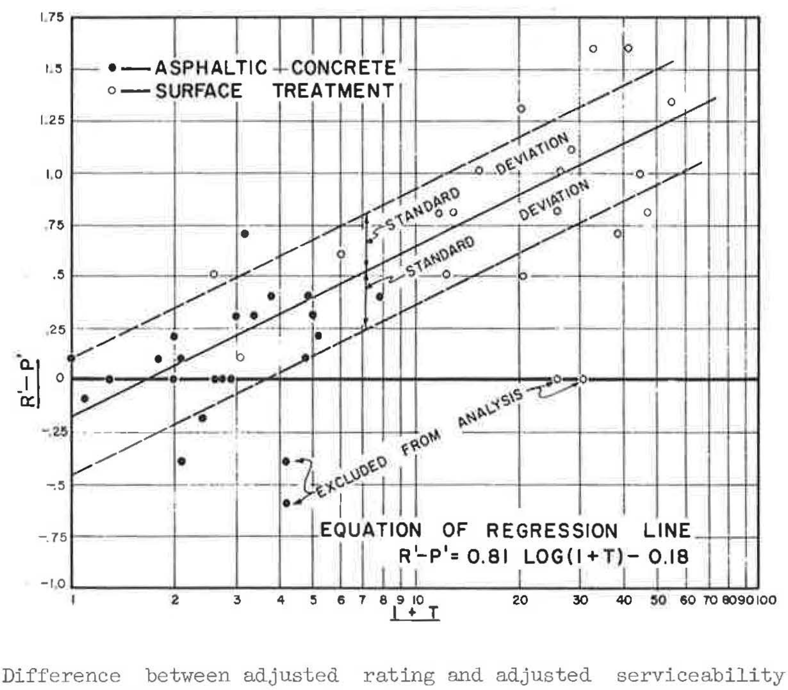

Figure 6. Difference between adjusted rating and adjusted serviceability index as a function of roughness of surface texture. Data are from Table 2.

$$R' = p' - 0.18 + 0.81 \log(1 + T) \quad (6)$$

in which p' is found from Eq. 1, and $\overline{SV}$ is the (CHLOE slope variance - 2.5) × 10^6.

Let p_c = the serviceability index corrected for surface texture.

Then according to Eq. 6

$$p_c = 4.85 - 1.91 \log(1 + \overline{SV}) + 0.81 \log(1 + T) -$$

$$0.01\sqrt{C + P} - 1.38\overline{RD}^2 \quad (7)$$

in which $\overline{SV}$ is the (CHLOE slope variance - 2.5) × 10^6 and all other terms are as previously defined.

According to the analysis of the adjusted data, Eq. 7 predicts the adjusted ratings (actual rating + 10%) with a root mean square residual of 0.28. This error compares favorably with the error of 0.38 reported for Eq. 1 in the reference previously cited, but would have been increased somewhat had the four sections eliminated from the analysis been included.

Values of p_c computed from Eq. 7 are given in the next to last column of Table 2. Prediction errors are shown in the last column.

The small study reported here is by no means conclusive. Experience at the Road Test indicated that the initial rating session for any rating panel may be somewhat erratic due to variations in rating technique as the study progresses. Since the analysis of this data, this equipment has subsequently participated in a nationwide correlation and rating session at Purdue University under the auspices of the NCHRP. The results of that test are presently being analyzed and will be reported, at which time it may be desirable to rerun this analysis to include that data. The arbitrary adjustment of the data must also be checked.

CONCLUSIONS

The following conclusions appear to be justified by the data and analysis presented here:

1. The local rating panel seemed to rate pavements at a slightly lower level than the AASHO Road Test rating panel. The tendency was most noticeable in the case of pavements in the "very good" category.
2. The texturemeter or a similar device is a necessary tool for use with the CHLOE profilometer on coarse-textured pavements such as surface treatments.
3. The modified formula for the serviceability index (Eq. 7) is believed to be satisfactory for the purposes of this project and should be used for calculating the serviceability index of flexible pavements in lieu of the original AASHO Road Test formula (Eq. 1), inasmuch as about one-half of the flexible pavement test sections have a coarse-textured surface.
4. The CHLOE profilometer in use on this project should again be correlated with the AASHO Road Test profilometer. If the correlation between the two instruments is found to have changed, it may be necessary to make corresponding changes in the coefficients of Eq. 7.

RECOMMENDATIONS

The authors realize that the instrument developed and discussed herein is empirical and that changes in its design and/or construction can affect the results. Three models of the texturemeter have already been built in Texas. Model 2 is the model described in Appendix B. A second instrument was constructed from these plans for the South Dakota Highway Department. Appendix C compares the output of the two instruments.

It is recommended that some measuring device such as the texturemeter be used in conjunction with the CHLOE profilometer if accurate PSI estimates are to be obtained. Additional modification and improvements of the present device are encouraged.

ACKNOWLEDGMENTS

The authors are indebted to James H. Aiken, District Engineer, District 9, and other engineers of the Texas Highway Department who participated in the work of rating the pavements, as well as to other members of the project staff.

The texturemeter was designed by staff members working in cooperation with Josef Budig, Scientific Instrument Maker, Research and Instrument Shop, Texas Engineering Experiment Station, who constructed the first experimental model. The model used in this study was constructed in the Texas Highway Department's Equipment Division shops under the supervision of Paul Hancock. W.R. Hudson of the Highway Department contributed valuable suggestions leading to improvements in the second model.

Members of the rating panel were J.E. Kelly, S.P. Gilbert, G.C. Cleveland, John Neubauer, R.E. Burns, W.W. Miller, and John Nichols, all of District 9 of the Highway Department; W.R. Hudson, John Nixon, and P.B. Rapstine of the Highway Design Division; F.H. Scrivner and E. L. Hlavaty of the project staff.

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Appendix A

RELATION OF TEXTUREMETER READING TO SLOPE VARIANCE

Because the texturemeter is designed to provide a correction to be applied to the slope variance measured by the CHLOE profilometer, it is of interest to investigate the relationship between slope variance and the texturemeter reading.

When the texturemeter is placed in contact with a pavement surface, the string takes a shape which approximates the shape of the surface (Fig. 7). The dial gage of this 5-probe texturemeter would measure the difference between the straight-line distance, AB, and the total length of the broken line. That is, the dial would read the difference, Δ, in

$$\Delta = h_1 + h_2 + h_3 + h_4 - AB \tag{8}$$

In the more general case of a texturemeter with n + 1 probes,

$$\Delta = \sum_{i=1}^{n} h_i - AB \tag{9}$$

The corresponding slope variance is calculated as follows:

Let θ_i be the angle between the string segment connecting probe (i - 1) and probe i. The slope of this segment is tan θ_i. The variance, σ^2, is

$$\sigma^2 = \left(\frac{1}{n - 1}\right)\left[\sum_{i=1}^{n} \tan^2 \theta_i + \frac{1}{n} \sum_{i=1}^{n} \tan \theta_i\right] \tag{10}$$

In Figure 7 it may be seen that

$$\tan \theta_3 = \frac{a_3}{b} \tag{11}$$

in which $a_3 = Y_3 - Y_2$, and b is the spacing of the probes.

In general, then

$$\tan \theta = \frac{Y_i - (Y_i - 1)}{b} \tag{12}$$

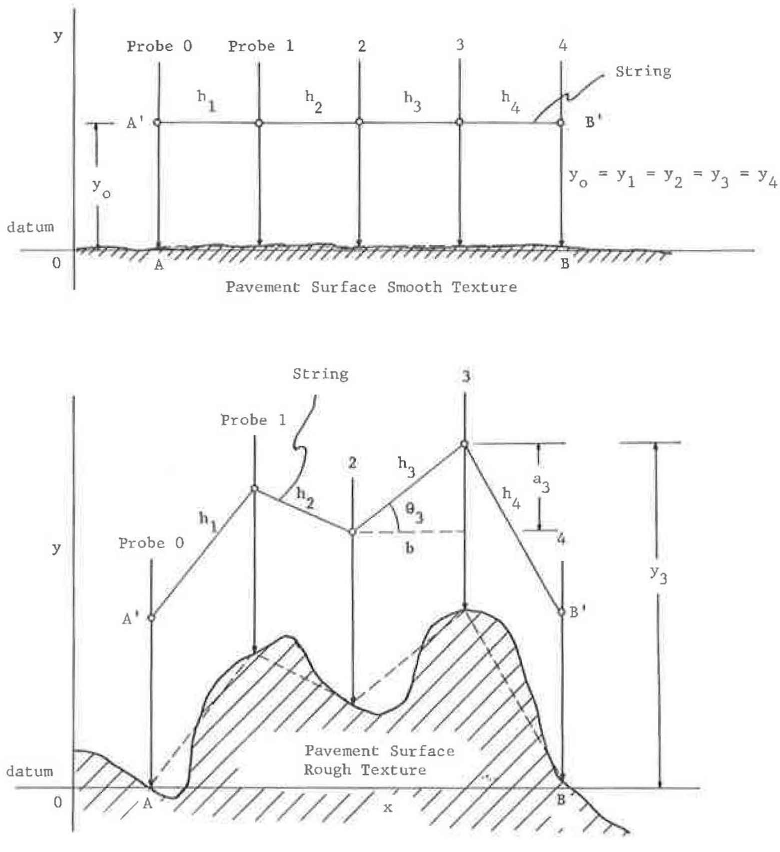


Figure 7. Schematic of a 5-probe texturemeter. The string, parallel to the dashed lines connecting the probe points, approximates shape of pavement surface.

and

$$\sum_{i=1}^{n} \tan \theta_i = \frac{1}{b} \sum_{i=1}^{n} (Y_i - Y_{i-1}) \tag{13}$$

For the 5-probe texturemeter (Fig. 7), n = 4 and

$$\sum_{i=1}^{4} (Y_i - Y_{i-1}) = Y_1 - Y_0 + Y_2 - Y_1 + Y_3 - Y_2 + Y_4 - Y_3 = Y_1 + Y_4$$

But $Y_1 = Y_4 = 0$, as may be seen in Figure 7. Therefore

$$\sum_{i=1}^{4} (Y_i - Y_{i-1}) = 0$$

Then in the general case

$$\sum_{i=1}^{n} (Y_i - Y_{i-1}) = 0 \tag{14}$$

According to Eqs. 10 and 14, the slope variance is

$$\sigma^2 = \frac{\sum_{i=1}^{n} \tan^2 \theta_i}{n-1} \tag{15a}$$

From Eqs. 12 and 15 it is seen that

$$\sigma^2 = \frac{\sum_{i=1}^{n} (Y_i - Y_{i-1})^2}{b^2 (n-1)} \tag{15b}$$

Now $(Y_3 - Y_2)^2 = h_3^2 - b^2$, as may be seen by reference to the right triangle shown in Figure 7. Therefore, in the general case

$$(Y_i - Y_{i-1})^2 = h_i^2 - b^2$$

Therefore

$$\sigma^2 = \frac{\sum_{i}^{n} h_i^2}{b^2 (n-1)} - \frac{n}{n-1} \tag{16}$$

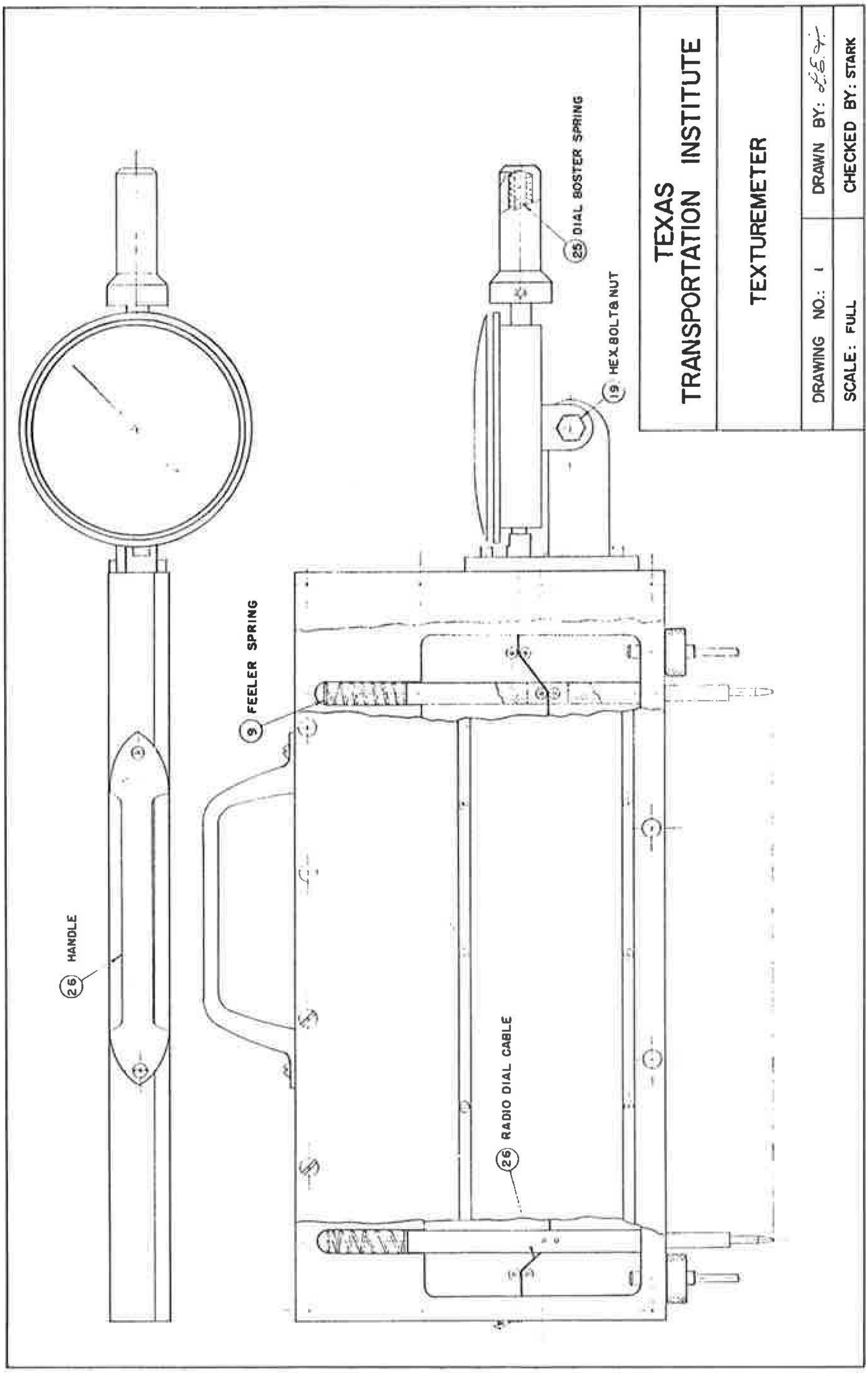
By comparing the slope variance, σ^2, given in Eq. 16, with the texturemeter reading, Δ, given in Eq. 9, it can be seen that both quantities are functions of the variable, h. Therefore, a correlation exists between the slope variance of the pavement surface from point A to point B, and the reading of the texturemeter in the same area.

Appendix B

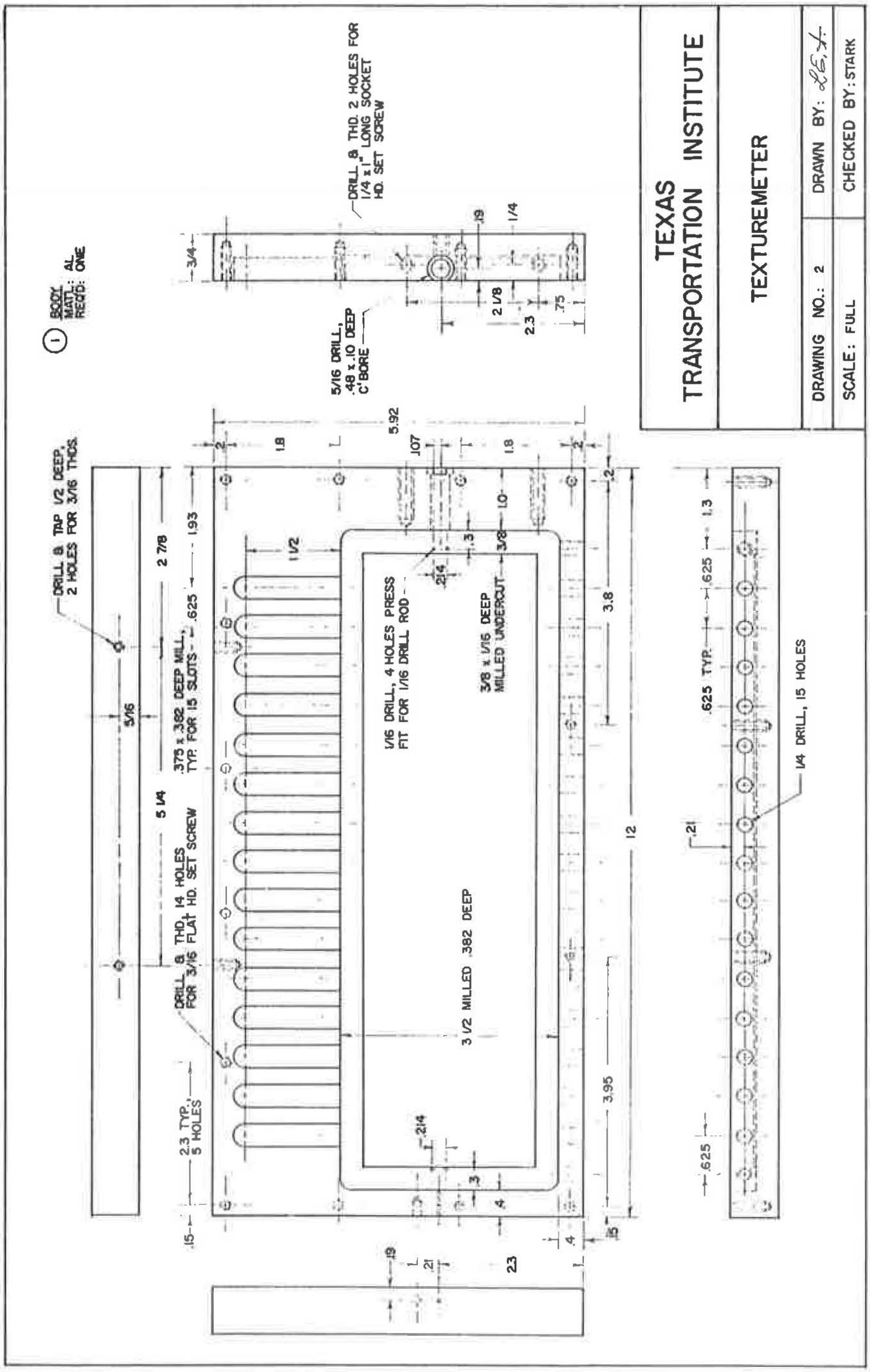
PLANS AND PARTS LIST FOR TEXAS TEXTUREMETER MODEL 2

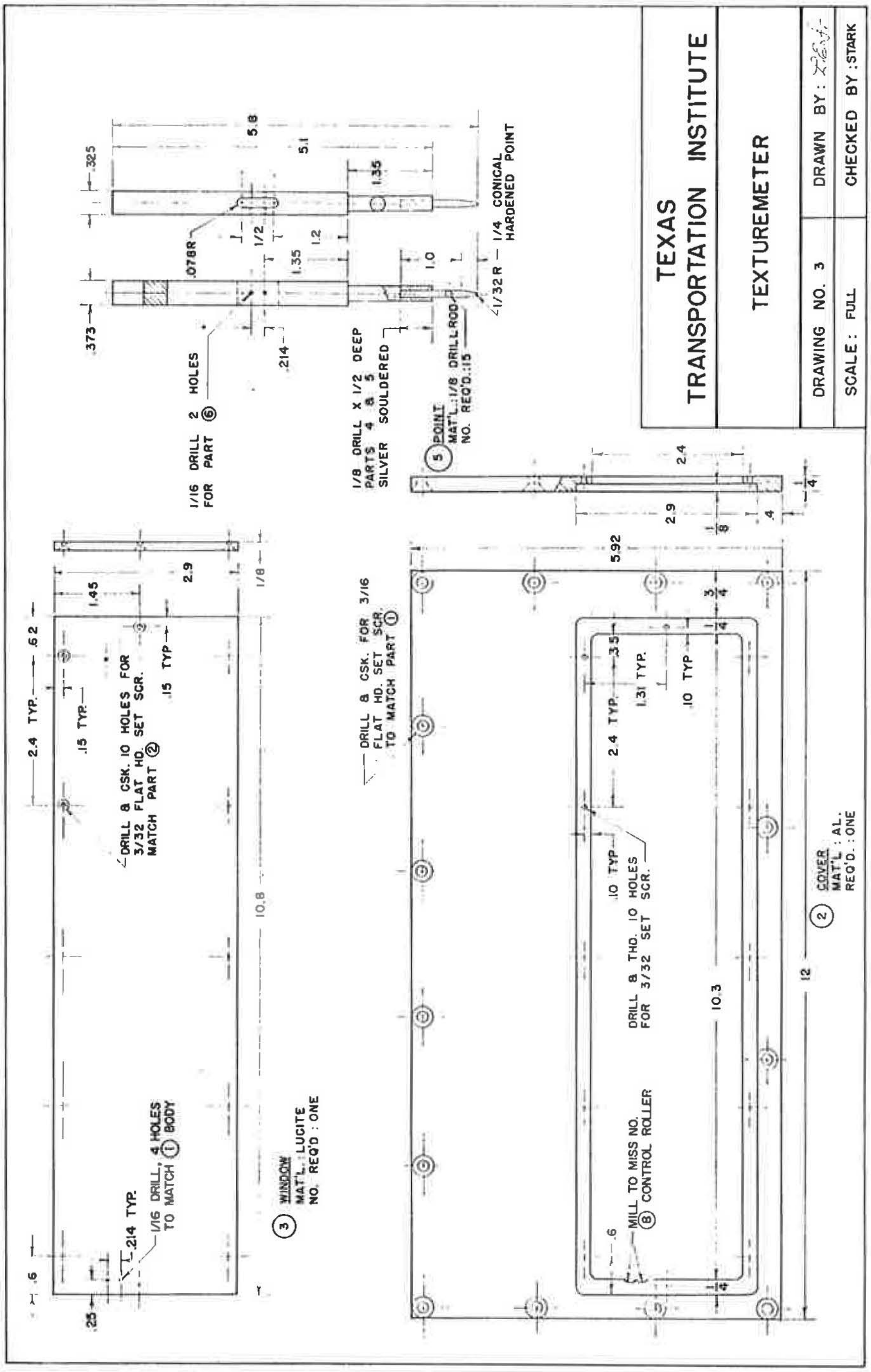
PARTS LIST

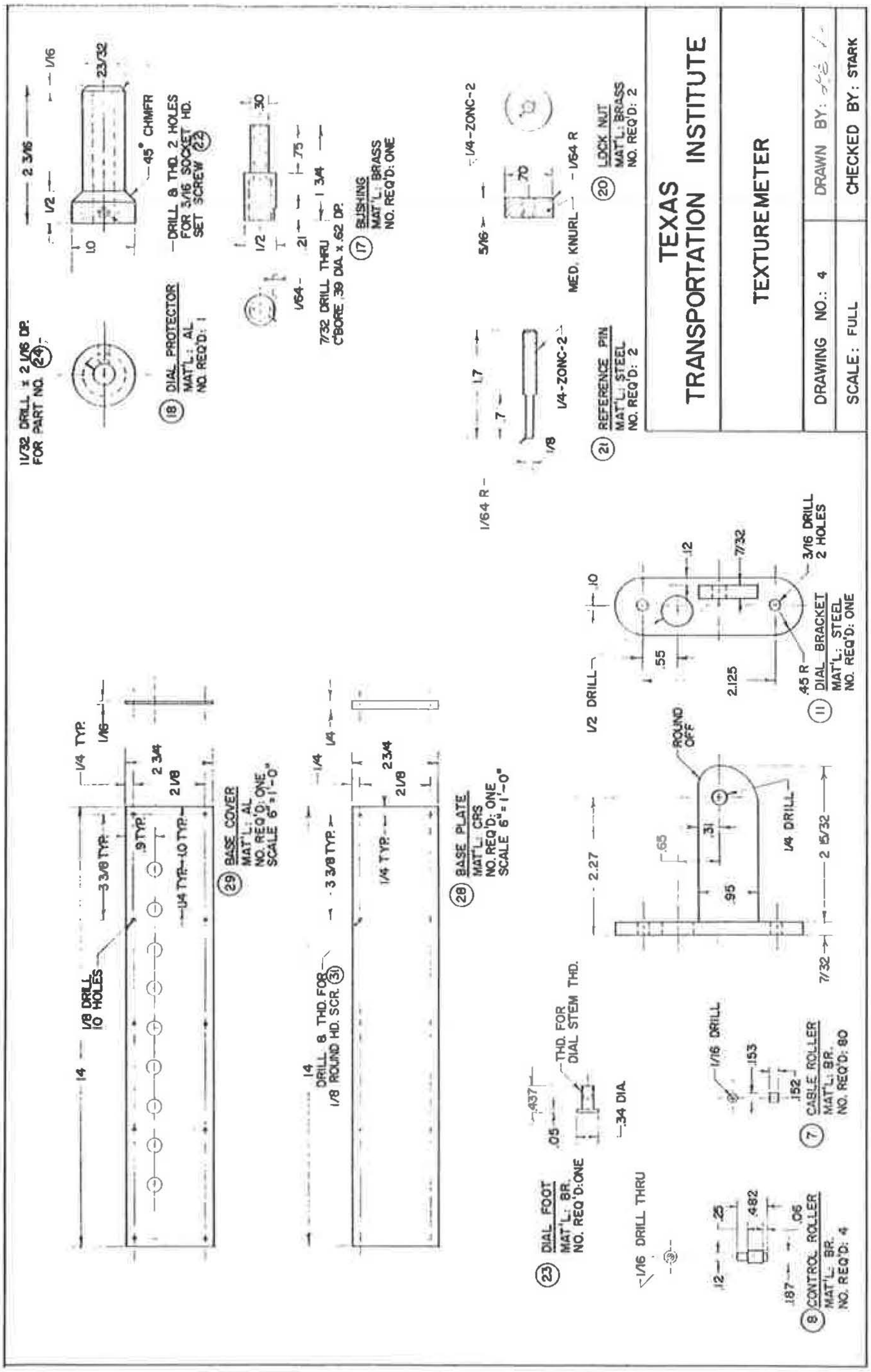
Part No.	Name	No. Required	Material
1	Body	1	AL
2	Cover	1	AL
3	Window	1	PL
4	Feeler	15	Brass
5	Point	15	Steel
6	Drill Rod, $\frac{1}{16} \times 0.325$	30	Steel
7	Cable Roller	30	Brass
8	Control Roller	4	Brass
9	Feeler Spring	15	0.314 O.D., 0.023 Gage
10	Ames Dial No. 482	1	
11	Dial Bracket	1	Steel
12	$\frac{3}{32} \times \frac{1}{8}$ Flat HD Set SCR	10	BR
13	$\frac{3}{16} \times \frac{1}{2}$ Flat HD Set SCR	14	BR
14	$\frac{1}{8} \times \frac{3}{8}$ Round HD SCR	1	BR
15	$\frac{3}{16} \times \frac{1}{4}$ Round HD SCR	2	BR
16	$\frac{3}{16} \times 1''$ Allen SCR Knurled	2	ST
17	Bushing	1	BR
18	Dial Protector	1	AL
19	$\frac{1}{4} \times \frac{1}{2}$ HEX HD Bolt & Nut	1	ST
20	Lock Nut Knurled	2	BR
21	Reference Pin	2	ST
22	$\frac{3}{16} \times \frac{1}{8}$ Set SCR	2	ST
23	Dial Foot	1	BR
24	Dial Booster Spring	1	0.245 O.D., 0.022 Gage
25	Handle	1	BR
26	0.025 Radio Dial Cable	1	No. 75A - 100 Nylon
27	Drill Rod $\frac{1}{16} \times \frac{3}{4}$	4	ST
28	Base Plate	1	CRS
29	Base Cover	1	AL
30	$\frac{1}{8} \times \frac{1}{4}$ Round HD SCR	8	BR



TEXAS
TRANSPORTATION INSTITUTE
TEXTUREMETER
DRAWING NO.: 1
DRAWN BY:
SCALE: FULL
CHECKED BY: STARK
25 DIAL BOSTER SPRING
19 HEX. BOLT & NUT
9 FEELER SPRING
26 HANDLE
26 RADIO DIAL CABLE


TEXAS
TRANSPORTATION INSTITUTE
TEXTUREMETER
DRAWING NO.: 2
SCALE: FULL
DRAWN BY:
CHECKED BY: STARK
1 BODY
MATL: AL
REQ'D: ONE
DRILL & TAP 1/2 DEEP, 2 HOLES FOR 3/16 THDS.
DRILL & THD. 14 HOLES FOR 3/16 FLAT HD. SET SCREW
.375 x .382 DEEP MILL, TYP. FOR 15 SLOTS
2.3 TYP. 5 HOLES
1/16 DRILL, 4 HOLES PRESS FIT FOR 1/16 DRILL ROD
3/8 x 1/16 DEEP MILLED UNDERCUT
3 1/2 MILLED .382 DEEP
5/16 DRILL, .48 x .10 DEEP C'BORE
DRILL & THD. 2 HOLES FOR 1/4 x 1" LONG SOCKET HD. SET SCREW
1/4 DRILL, 15 HOLES
.625 TYP.


TEXAS TRANSPORTATION INSTITUTE
TEXTUREMETER
DRAWING NO. 3
DRAWN BY:
SCALE: FULL
CHECKED BY: STARK
1/16 DRILL 2 HOLES FOR PART 6
.078R
1/8 DRILL X 1/2 DEEP PARTS 4 & 5 SILVER SOULDERED
5 POINT MAT'L.:1/8 DRILL ROD NO. REQ'D.:15
1/32 R
1/4 CONICAL HARDENED POINT
DRILL & CSK. 10 HOLES FOR 3/32 FLAT HD. SET SCR. MATCH PART 2
1/16 DRILL, 4 HOLES TO MATCH 1 BODY
3 WINDOW MAT'L.: LUCITE NO. REQ'D: ONE
DRILL & CSK. FOR 3/16 FLAT HD. SET SCR. TO MATCH PART 1
DRILL & THD. 10 HOLES FOR 3/32 SET SCR.
MILL TO MISS NO. 8 CONTROL ROLLER
2 COVER MAT'L.: AL. REQ'D.: ONE


TEXAS
TRANSPORTATION INSTITUTE
TEXTUREMETER
DRAWING NO.: 4
SCALE: FULL
DRAWN BY:
CHECKED BY: STARK
18 DIAL PROTECTOR
MAT'L: AL
NO. REQ'D: 1
11/32 DRILL x 2 1/16 DP. FOR PART NO. 24
DRILL & THD. 2 HOLES FOR 3/16 SOCKET HD. SET SCREW 22
45° CHMFR
17 BUSHING
MAT'L: BRASS
NO. REQ'D: ONE
7/32 DRILL THRU
C'BORE .39 DIA x .62 DP.
21 REFERENCE PIN
MAT'L: STEEL
NO. REQ'D: 2
1/4-20NC-2
20 LOCK NUT
MAT'L: BRASS
NO. REQ'D: 2
MED. KNURL
1/64 R
11 DIAL BRACKET
MAT'L: STEEL
NO. REQ'D: ONE
1/2 DRILL
3/16 DRILL 2 HOLES
.45 R
ROUND OFF
1/4 DRILL
29 BASE COVER
MAT'L: AL
NO. REQ'D: ONE
SCALE 6" = 1'-0"
1/8 DRILL 10 HOLES
28 BASE PLATE
MAT'L: CRS
NO. REQ'D: ONE
SCALE 6" = 1'-0"
DRILL & THD. FOR 1/8 ROUND HD. SCR. 31
23 DIAL FOOT
MAT'L: BR.
NO. REQ'D: ONE
THD. FOR DIAL STEM THD.
.34 DIA
7 CABLE ROLLER
MAT'L: BR.
NO. REQ'D: 80
1/16 DRILL
8 CONTROL ROLLER
MAT'L: BR.
NO. REQ'D: 4
1/16 DRILL THRU

Appendix C

COMPARISON OF TWO TEXTUREMETERS

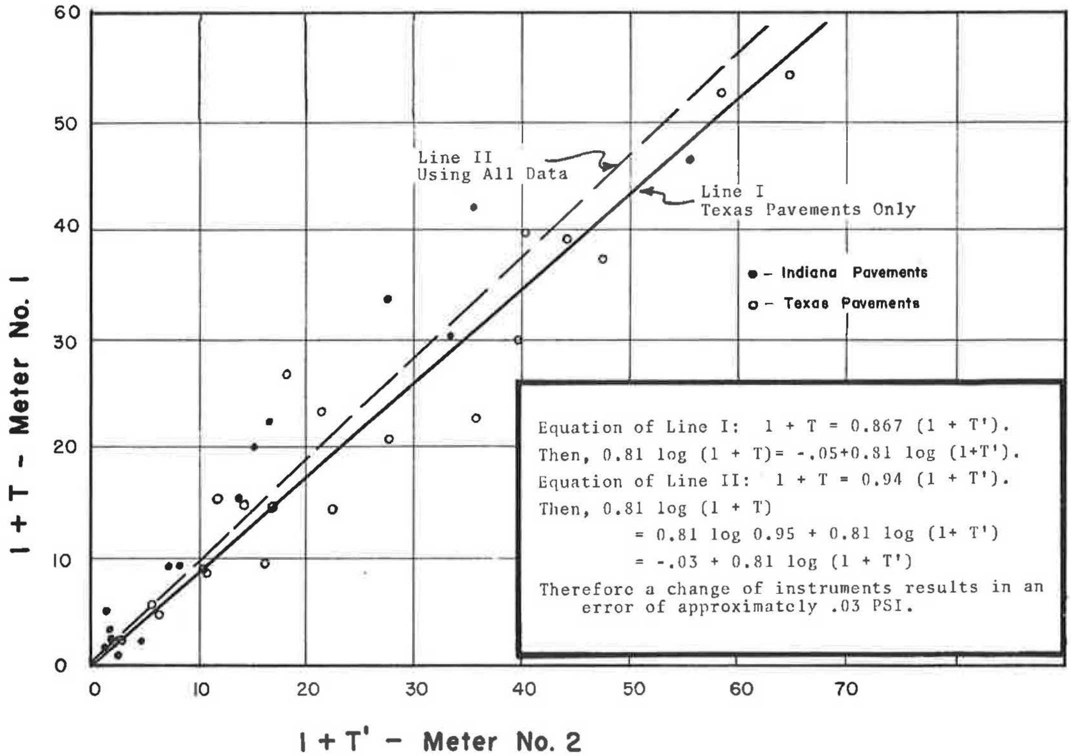


Figure 8. Comparison of two texturemeters.

Can Dynamic Tire Forces Be Used as A Criterion of Pavement Condition?

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A moving vehicle exerts a fluctuating component of force on the highway due to vertical motion induced in the vehicle by unevenness of the pavement. This force, superimposed on the static wheel load, is frequently called the dynamic wheel force or the dynamic tire force.

This paper includes a brief description of instrumentation used to determine the dynamic tire force by measuring the fluctuation of the air pressure in a tire of a moving vehicle. Actual force records that were taken on several sections of pavement are included. From these records the corresponding root mean squared value of the dynamic force is determined, and the possible use of this quantity as a criterion of pavement condition is suggested. In addition, for certain pavement sections the frequency of occurrence of various magnitudes of the dynamic tire force is determined.

The influence of speed on dynamic force is investigated by operating the test vehicle over the same length of pavement at different velocities. In like manner the influence of tire inflation pressure on dynamic force is studied by conducting tests in which only the inflation pressure was varied.

The significance of these parameters relative to a pavement criterion based on dynamic tire force is discussed.

•A VEHICLE moving over a perfectly smooth pavement will in general exert only the static wheel loadings on the highway. Unevenness in the pavement profile will, however, induce vertical motion in the vehicle and this will produce fluctuating force components that will be superimposed upon the static wheel loads. These fluctuating force components are referred to in this paper as the dynamic tire forces. The question is raised as to whether or not they can serve as a criterion of pavement condition.

It is evident that the dynamic tire force is a criterion of the interaction between a vehicle and the pavement. On a perfectly smooth pavement the dynamic tire force would be zero, whereas on a rough pavement the force would be large. Unfortunately, however, this force is influenced by factors other than pavement roughness. The suspension characteristics of the vehicle are significant as well as the speed of the vehicle. It has been shown (1) that under certain conditions it is possible to induce either large or small dynamic forces by simply varying the speed of the vehicle. Evidently, if a comprehensive study is made of the dynamic tire force, the vehicle characteristics and the vehicle velocity must be considered along with the pavement profile. Conversely, if only the effect of pavement condition on the dynamic tire force is to be studied, then tests must be made with the same vehicle at the same speed over different pavement sections. These factors were investigated in the test reported herein, and the

Paper sponsored by Committee on Pavement Condition Evaluation.

results gave rise to the possible use of dynamic force as a criterion of pavement condition.

The question can well be raised as to the effect of the dynamic tire forces on the highway. Under certain conditions these forces may be small relative to the static wheel loads and they are applied for a relatively short period of time at any one location on the highway. The response of the highway to these forces, however, has been a matter of interest to many investigators.

Although only small deformations may occur in the highway at moderate distances from the point of application of a dynamic tire force, this force may not be insignificant as far as damage to the highway is concerned. High forces will result in high contact stresses, and rapid surface deterioration may result even though the interior structure of the highway may not be adversely affected by such a force. Therefore, it is evident that the relationship between dynamic tire force and highway response is an important area for continued study.

In like manner, the question as to the effect of the dynamic tire forces on the vehicle can also be raised. These forces will in general have large vertical components that will give rise to deformations of the tires of the vehicle. Depending on the frequency with which these forces are applied, it is possible to have either large or small motions induced in the sprung mass (body) of the vehicle. If large body motions are induced, the passengers may experience considerable discomfort. If very little body motion is induced, the passengers may be unaware of the fact that large dynamic forces are acting on the tires of the vehicle.

It is evident that passengers, riding in a vehicle, may be unaware of large forces that are generated between the pavement and the vehicle even though these forces may be causing damage to the pavement. These passengers may even be giving an excellent rating to the pavement at the instant that damage is resulting from an unfavorable combination of pavement condition, vehicle velocity and vehicle suspension characteristics.

Is a pavement in satisfactory condition when it will induce large dynamic tire forces? Is a passenger capable of evaluating pavement conditions based solely on the "ride" of the vehicle? Clearly the task of evaluating pavement condition is not an easy one.

In this investigation the dynamic tire force was measured continuously as a test vehicle moved along the pavement, and the resulting force records were analyzed to obtain the root mean square value of the dynamic tire force. In certain cases the frequency of occurrence of certain magnitudes of the force was determined. The effects of vehicle velocity and tire inflation pressure on the dynamic force were briefly investigated.

MEASUREMENT OF TIRE FORCE

The measurement of the dynamic tire forces has been of interest to many investigators. The Bureau of Public Roads conducted an investigation into this problem and described a system for measuring dynamic wheel reactions that utilized the fluctuating air pressure in a tire of a vehicle (2). The technique was further developed by the Michigan State Highway Department Laboratories in Lansing. The force measuring system used to obtain the results described in this paper was virtually identical to that used in Michigan except for minor modifications. An extensive investigation into methods for measuring dynamic wheel forces is at present being conducted in Germany (3).

The relationship between the dynamic tire force and the change in tire pressure is shown in Figure 1. The tire air pressure, p, is actually the change in the inflation pressure due to the motion of the vehicle on the highway. With this change in air pressure is an associated change in the force of the tire on the highway as indicated by F. By monitoring the air pressure in a tire it is possible to measure the dynamic wheel forces that are superimposed on the static wheel load.

In making tire pressure measurements, the valve core was removed from the valve stem of the tire and a short tube was connected from the valve stem to the rotating element of a special seal that was mounted on the wheel of the vehicle. From the stationary element of this seal, a tube was connected to the pressure measuring system

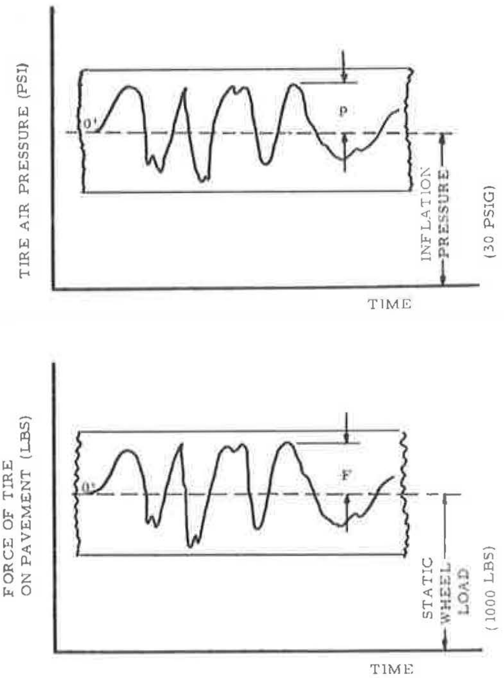

Figure 1. Relationship between tire air pressure and tire force.

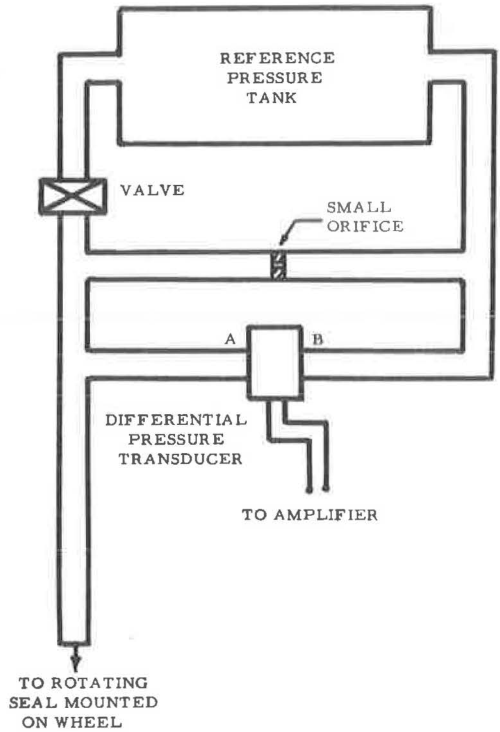

Figure 2. System for measuring change in tire air pressure.

shown schematically in Figure 2. By using this seal, it was possible to monitor the pressure in a rotating tire.

A schematic diagram of the system for measuring the change in the air pressure in the tire is shown in Figure 2. Before making the measurements, the valve to the reference pressure tank was opened, establishing the static tire pressure in all parts of the pressure measuring system. In taking the pressure measurements the valve was closed, thus subjecting side A of the differential pressure transducer to the fluctuating pressure in the tire while at the same time subjecting side B to the original tire pressure established in the reference pressure tank. As the vehicle moved down the highway, the pressure transducer responded to the difference in pressure between these two values and transmitted this information in the form of an electrical signal to an appropriate electronic circuit in which the signal was amplified and recorded with an oscillograph.

Certain difficulties are encountered in making measurements with this system. Air pressure in a tire is sensitive to temperature, and any heating or cooling of the tire will produce a difference in pressure relative to the original tire pressure introduced into the reference tank. In addition, small leaks will also cause a differential pressure to exist even though the vehicle may not be in motion. As a consequence, it is evident that undesirable factors may influence the dynamic tire pressure measurements when using this system.

Relatively slow changes in tire pressure can be eliminated by the introduction of an appropriately selected orifice as shown by the dotted line in Figure 2. With this modification the effect of heating may be minimized, but the selection of the proper orifice requires care so that no appreciable distortion of more rapid pressure changes will result.

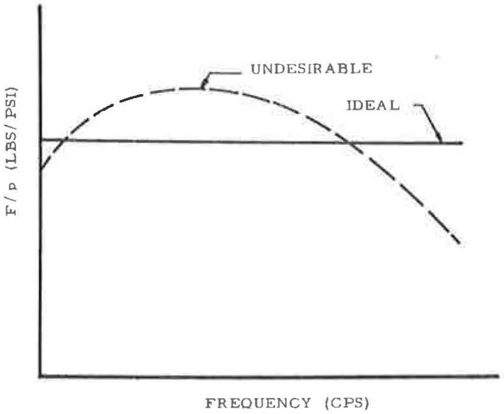

Figure 3. Calibration curves for pressure measuring system.

It is also essential that the proper tube lengths and volumes be selected, otherwise resonances will occur due to the vibration of the air in the system. These vibrations will influence the differential pressure record and produce inaccuracies in the results. The factors that influence the behavior of this system are currently being studied, and detailed information concerning the design and behavior of this equipment will be available in a later report if there is sufficient interest.

Because it is necessary to convert fluctuating air pressure measurements to those of force acting on the tread of the tire, an appropriate calibration relationship must be obtained. If the system is not carefully designed, this relationship will vary considerably with frequency. Relationships between the dynamic tire force acting on the tread of the tire and the change in air pressure as a function of frequency are shown in Figure 3. If inappropriate parameters are selected, an undesirable relationship will be obtained as indicated. This characteristic, shown by the dotted line, indicates that the ratio of tire force to tire pressure varies with frequency. Thus at certain frequencies higher forces will result in the same change of tire pressure. This means that records taken in the time domain cannot easily be interpreted because different frequencies require different calibration factors to convert them to the appropriate value of tire force. The ideal characteristic is shown by the solid line in which the ratio of tire force to tire pressure is the same value at all frequencies. With this characteristic it is possible to take a time domain record, such as that shown in Figure 4, and to determine the force that will exist on the tire at a selected time, regardless of the frequencies that exist in the record. It is evident that a calibration relationship, shown by the solid line, is most desirable, and hence the pressure measuring system shown in Figure 2 should be designed with this objective in mind.

The pressure measuring system was connected to the right front wheel of the test vehicle which was operated at a constant velocity over various sections of pavement. Knowing the velocity of the vehicle, it is possible on the analog record to indicate the position of the vehicle on the highway and to determine the corresponding tire force. In making the actual record, it is necessary to switch on the electronic equipment and to approach the test section with the pressure measuring equipment in operation. To indicate the location on the record of the start of the test section, a special device is placed on the highway at the beginning and at the end of the selected section of pavement. When the tire of the test vehicle strikes this device, a special mark is made on the record establishing the beginning and end of the pressure measurements relative to the highway.

Records obtained on the oscillograph are actually records of fluctuating tire pressure versus time. By using the vehicle velocity it is possible to convert time to position of vehicle on the test section, and by using the calibration curve it is possible to convert tire pressure to tire force. To obtain accurate records it is necessary to calibrate the test vehicle before any change in vehicle characteristics can occur between the time of calibration and the taking of the record.

This system for measuring dynamic tire force must be carefully designed so as to avoid spurious pressure fluctuations and to obtain a constant calibration relationship. When these conditions are realized, this system is very sensitive and will give consistent results.

ANALYSIS OF TIRE FORCE RECORDS

Oscillograph records of tire pressure versus time for two extremes of pavement condition are shown in Figure 4. The number o′ on each record can be used to locate the records relative to the axes shown in Figure 1.

The ordinates representing change in air pressure can be converted to tire force if they are multiplied by the appropriate calibration factor previously mentioned. When this is done the force scale, shown at the left, can be added to each record. It is immediately evident that the difference in the smoothness of the two pavement sections has caused a large difference in the tire forces exerted by the wheel of the test vehicle on the highway. In both cases the same vehicle was operated at the same speed, thus the difference is due to the condition of the pavement.

At the beginning of each test section a special mark was recorded. Knowing the vehicle velocity and the elapsed time, to indicate the distance traveled from the starting point by the distance scale shown on the record, the tire force at any location can be determined.

Although the record for the smooth pavement shown in Figure 4 is useful in making a visual comparison of the two pavements in question, it is virtually useless for further study. It was necessary to obtain an additional record for which greater amplification in the recording equipment was used to obtain measurable results. This, however, changed the calibration factor for this record.

Having these records it is also possible to determine how often certain magnitudes of the dynamic tire force occur when driving over the pavement section. This can be done by first reading values of force from the record at equal intervals of distance. These are then assorted such that the number of values in various ranges of tire force

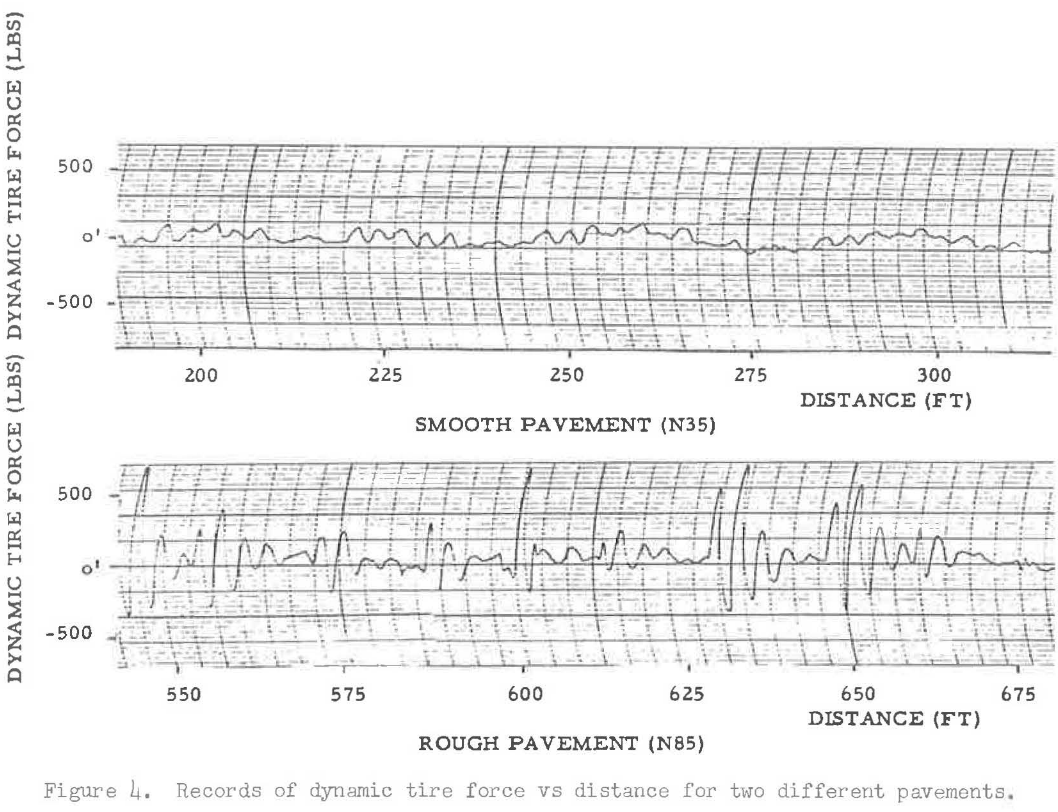

Figure 4. Records of dynamic tire force vs distance for two different pavements.

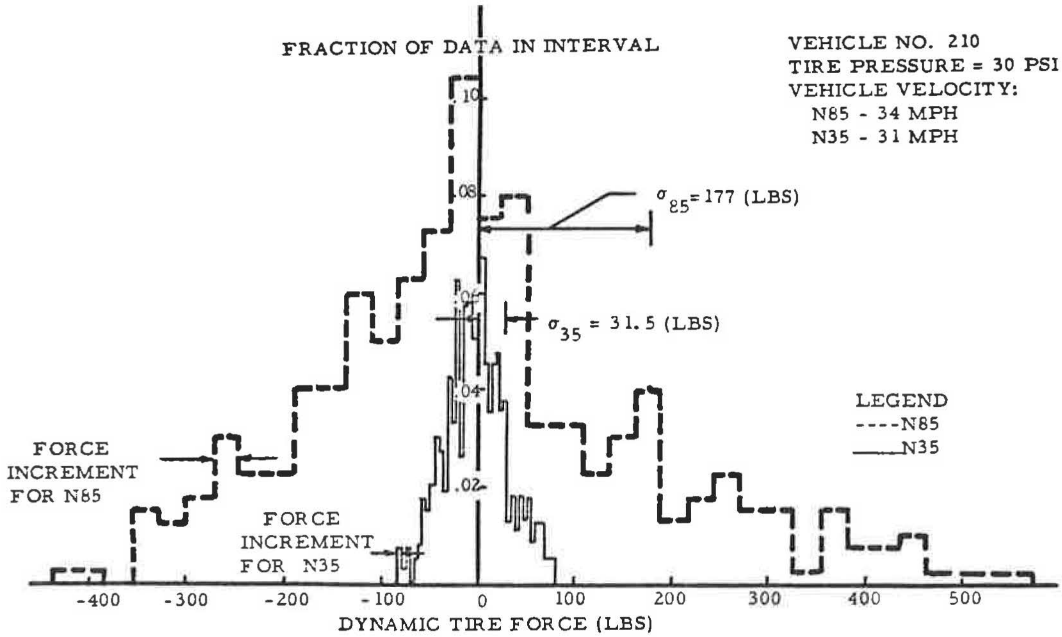

Figure 5. Distribution of dynamic tire forces for pavement sections N35 and N85.

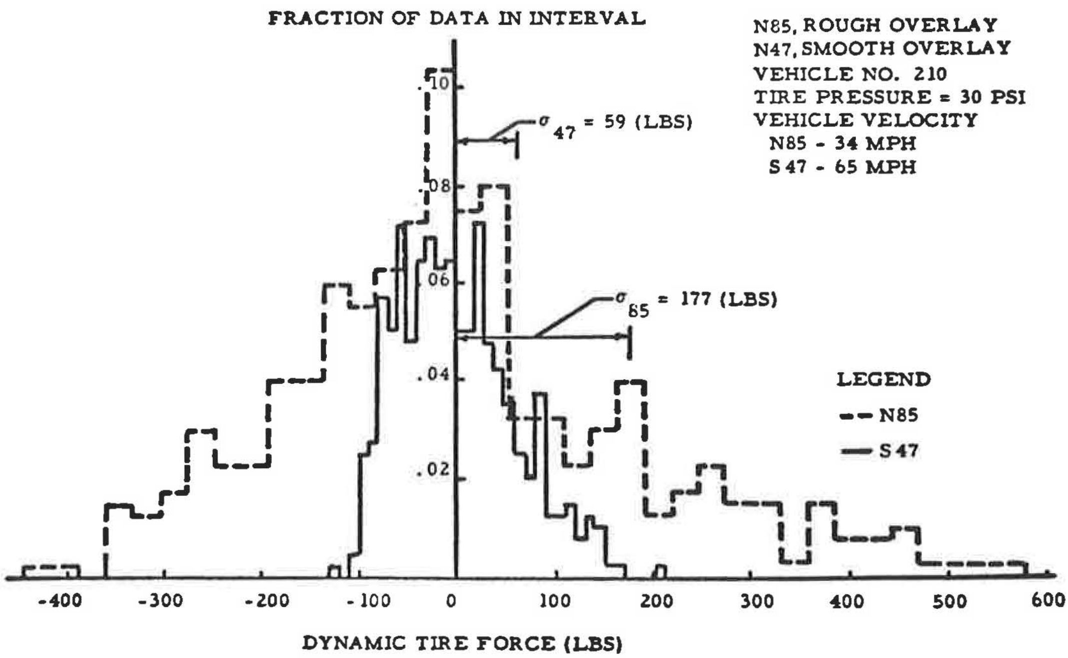

Figure 6. Distribution of dynamic tire forces for pavement section S47 and N85.

is determined. This number is divided by the total number of force readings included in the analysis of the record to obtain the fraction of the values in the force interval under consideration.

Figure 5 indicates results of this procedure when applied to the force records of which small sections are shown in Figure 4; distribution of dynamic forces in both can be compared. A much smaller increment of force must be used to study distribution of forces resulting from the smooth pavement than is used for rough pavement, otherwise no meaningful distribution can be obtained.

Although the pavements compared in Figure 5 are of different types of construction, there is as yet insufficient evidence to associate different force distributions with different types of construction. However, a comparison of pavements of the same type of construction but of different conditions of smoothness is shown in Figure 6, in which less striking differences are indicated.

It is evident from Figures 5 and 6 that the distribution of the dynamic tire force is not symmetrical. Larger positive tire forces are encountered than are those of negative sign. Because the positive values are added to the static wheel load to obtain the total tire force, it is clear that the assumption of a symmetrical distribution will underestimate the maximum forces that act on a highway. This possibility is investigated later in this paper.

It is, however, desirable to obtain a single summarizing statistic that can be used to describe the records in question. Inasmuch as both positive and negative forces are encountered, an average value of force is not significant. A better criterion can be obtained by squaring each force value obtained from the record, obtaining the sum of these values, dividing this sum by the total number of data values and taking the square root of the quotient. This statistic is called the root mean square (RMS) values of the tire force.

This was done for force records taken on several pavements that varied in condition as well as in type of construction. The results of these calculations are shown in Figure 7. The RMS force values for each type of pavement construction are recorded and grouped as shown. In addition, each pavement was assigned one of three subjective ratings (smooth, average, rough) as determined by the individuals who operated the test vehicle. Dynamic forces (RMS) ranging from 32 to 341 lb were obtained, resulting in a large range of values.

Although the subjective estimates of pavement condition are very crude, it is evident that low values of force are related to excellent pavement condition and that large values of force indicate that a considerable amount of roughness is present. Although a more extensive study of the correlation of dynamic tire force with pavement condition is necessary before final conclusions can be reached, it appears that pavement roughness is related to dynamic tire force.

As mentioned previously, the velocity of the test vehicle influences the dynamic tire forces. This was investigated by operating the test vehicle over the same length of pavement at four different velocities. The resulting RMS values of tire force are plotted against vehicle velocity (Fig. 8). Under the conditions encountered in this test, the tire force increases with velocity. This has been predicted theoretically from highway and vehicle characteristics (4, 5).

The distribution of the dynamic tire forces changed with vehicle velocity (Fig. 9). As the speed increased the maximum force increased. In addition, the frequency of occurrence of the larger forces also increased. This was accompanied by a small decrease in the negative forces but with a much larger frequency of occurrence of the negative forces. In other words, the skewness of the distribution increased with vehicle velocity.

The dynamic force distribution shown in Figure 9 by the solid lines (60-mph vehicle speed) was obtained on a pavement typical of many in use today. Likewise, the test vehicle velocity was close to that of many of the vehicles now using this section of pavement. This distribution is therefore not an extreme case such as that shown in Figure 5, but can be considered as representative of frequently encountered conditions.

The distribution is highly skewed, and to compare it with a normal distribution, a histogram was constructed based on a normal distribution having the same mean and the same RMS value as the skewed distribution (Fig. 10).

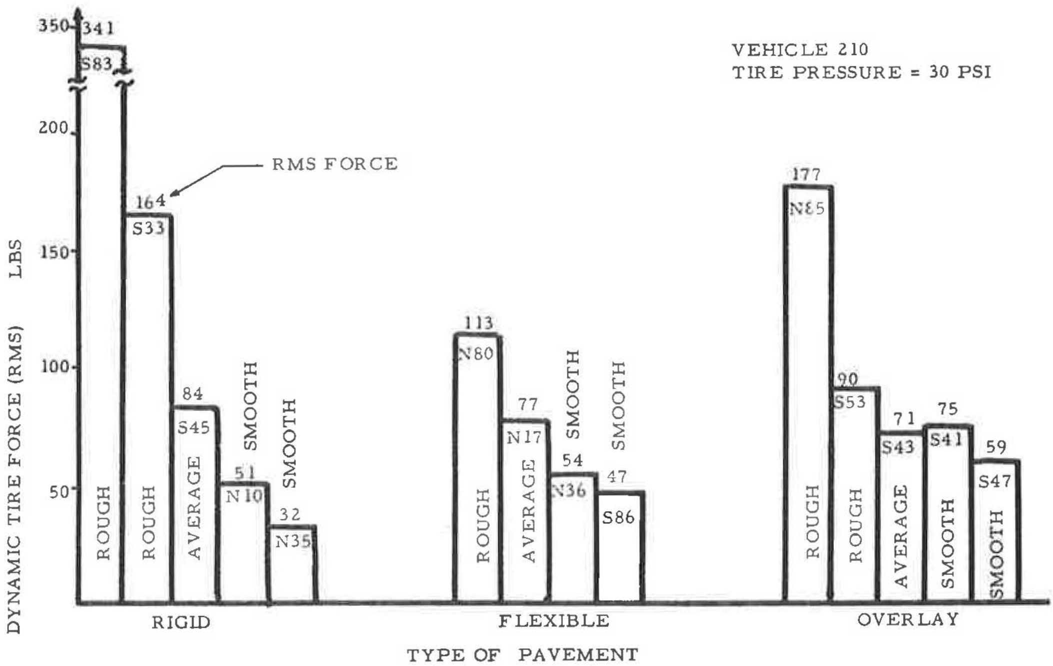

Figure 7. Root mean square value of dynamic tire force for different types of pavements.

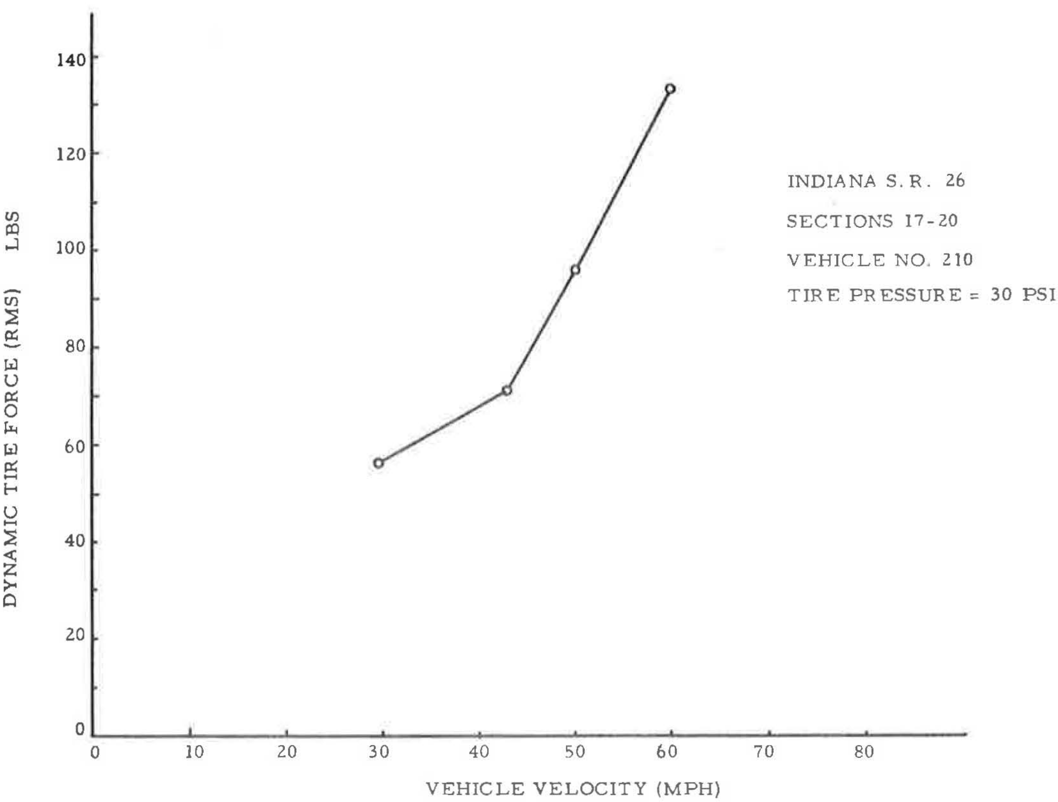

Figure 8. Root mean square value of dynamic tire force vs vehicle velocity.

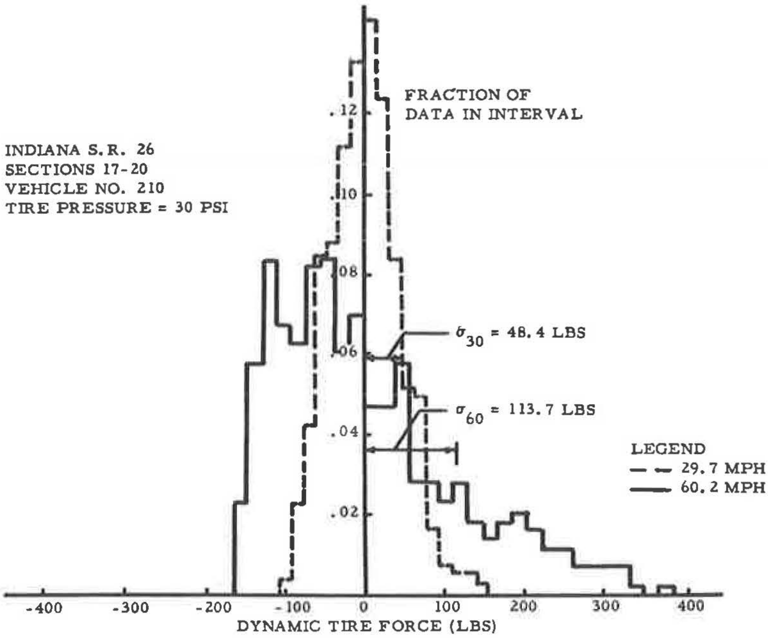

Figure 9. Distribution of dynamic tire forces at different speeds.

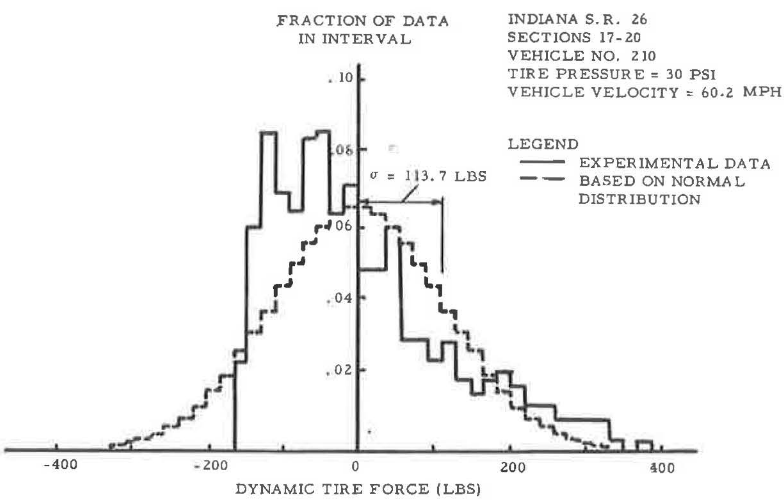

Figure 10. Comparison of dynamic tire force distributions.

Frequency of occurrence of large positive forces is underestimated by the assumption of normality, and the range of the negative forces is overestimated. Depending on the allowable error, the normal curve could be used as a first approximation for the positive forces. Inasmuch as these positive forces are added to the static wheel load to obtain the total force of the tire on the road, the frequency of occurrence of those forces that may be quite significant in the life of a highway will be underestimated. This error appears small, however, for data shown in Figure 10.

Changing the inflation pressure in the tire will influence the dynamic tire forces. This was investigated by operating the test vehicle over the same section of pavement at the same velocity but with different tire inflation pressures. Results are shown in Figure 11, in which the RMS value of the dynamic tire force plotted against tire inflation pressure.

Unfortunately tests were not conducted at relatively low inflation pressures and therefore this series of tests is incomplete. The curve indicates that low inflation pressures will result in lower tire forces, other factors remaining the same. A rapid drop in tire force could be expected for tire inflation pressures less than those recommended by the tire manufacturers (4). No such condition is shown in Figure 11, because all tests were conducted above the recommended pressure.

At high inflation pressures the calibration curve for the pressure measuring system approached that indicated as "undesirable" in Figure 3. A more accurate estimate of the RMS tire force requires a transformation in the frequency domain, and the procedure for determining force scales, described in connection with Figure 4, is no longer valid. An appreciable reduction in force will result from decreased inflation pressures, even though this is not indicated in Figure 11.

Vehicle characteristics influence the dynamic tire force records. The pavement will excite vertical motion in the suspension system of the vehicle that will consist largely of the natural frequencies of this system. This motion will be reflected in the tire pressure measurements.

A study of the frequencies in a tire force record can be made by using a power spectral density analysis (6). This analysis yields a curve, plotted as a function of frequency, the area under which gives the mean square value of the tire force. In addi-

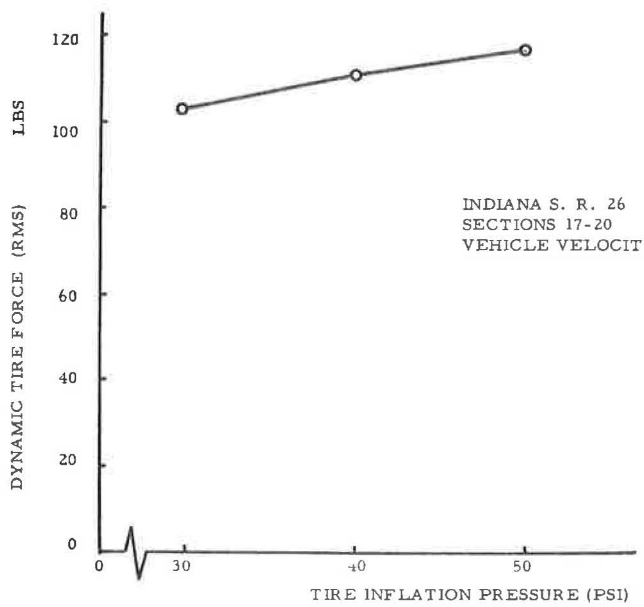

Figure 11. Dynamic tire force vs tire inflation pressure.

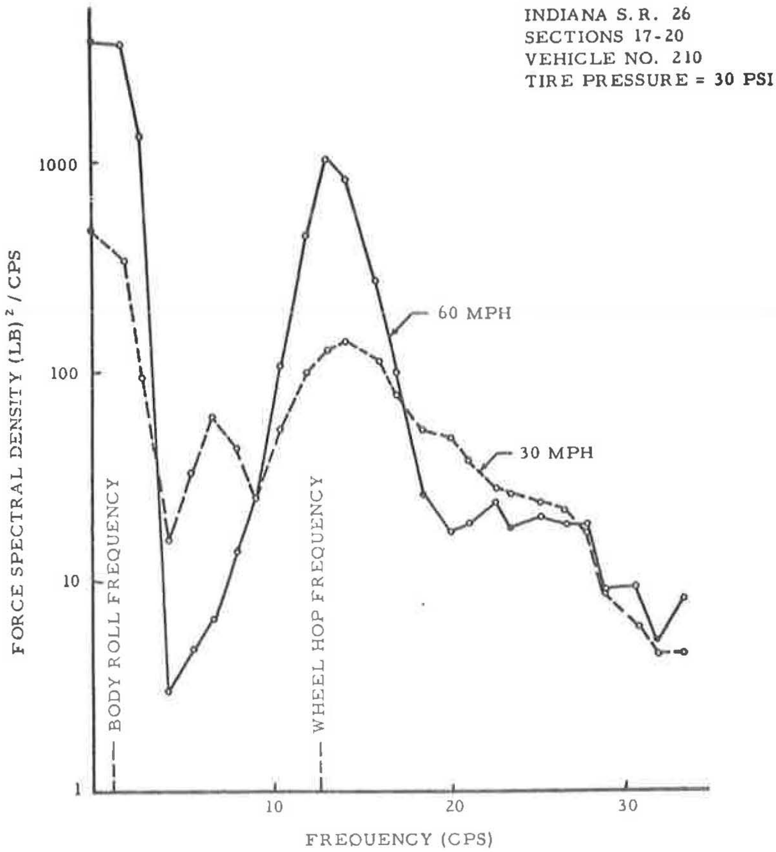

Figure 12. Power spectral density analysis of dynamic tire force records.

tion, the area bounded by any two ordinates gives the contribution to the mean square value that is made by those frequencies lying within the two ordinates.

The analysis can be useful in studying force records used to obtain distributions shown in Figure 9. The dynamic force power spectrum for the test vehicle traveling at 30 mph is shown by the dotted line in Figure 12. Two peaks of appreciable magnitude are indicated by this curve. The first peak, occurring over a range of low frequencies, indicates that the motion of the sprung mass (body roll) of the vehicle makes an appreciable contribution to the total mean square value of the dynamic tire force. A second peak, occurring over a range of higher frequencies, indicates the contribution to this value that results from motion of the unsprung mass of the vehicle (wheel hop).

Power spectral density function of the dynamic tire force for the same vehicle traveling over the same section of pavement at 60 mph is shown by the solid line in Figure 12. The area under this curve is greater, indicating the higher mean square force (Fig. 8).

In the change in the shape of the curve, the peak associated with low frequencies is higher, indicating that the motion of the sprung mass (body roll) changed appreciably. The motion of the unsprung mass (wheel hop) was also increased at the higher vehicle speed as evidenced by the large increase in the ordinates of the curve in the region of the wheel hop frequency.

Another change is in the power spectrum curve for 30 mph, in which a small intermediate peak can be seen approximately midway between the two large peaks. This is due to a large amount of excitation coming from the pavement because no natural frequencies exist in the suspension system of the test vehicle at this frequency. When the vehicle speed is doubled, the frequency of excitation from the highway is doubled. At 60 mph the excitation that caused the small intermediate peak has doubled in frequency, causing an additional excitation at the wheel hop frequency. Inasmuch as the vehicle is very responsive at this frequency, a great increase in tire force is produced.

SUMMARY

From the several techniques discussed, the most useful single statistic is the RMS value of the force. This can be used as a convenient summarization of a record and as a measure of the tire forces encountered for a selected pavement, vehicle and vehicle velocity.

Moreover, this quantity can be used to obtain a first approximation for frequency of occurrence of various magnitudes of total tire force exerted on the pavement. This can be done by using the static wheel load as the mean value of the total force, by assuming a normal distribution of the dynamic tire forces, and by using the RMS value of dynamic tire force as the standard deviation of the distribution curve.

The force power spectrum (Fig. 12) indicates extent to which various frequencies of vibration are present in the tire force records. Of greater importance, however, is the usefulness of this characteristic in performing operations in the frequency domain that cannot be performed readily in the time domain.

CONCLUSIONS

Dynamic tire force is related to pavement condition. Rough pavements cause large forces and smooth pavements cause relatively small forces. If the matter is pursued no further, the use of tire force as a criterion of pavement condition is very simple.

Unfortunately other aspects of the relationship between tire force and pavement condition must be considered. Confusion is introduced when different tire forces can be obtained on the same pavement by simply varying the speed of the vehicle. Can any pavement be rated as either good or bad, depending on the speed selected to take the tire force records? Can the rating of a pavement be changed from bad to good by simply changing the inflation pressure in the tires of the vehicle? Tire force as a pavement condition criterion is susceptible to these manipulations.

Is the actual significance of a pavement condition criterion solely a measure of the geometric properties of a pavement? If so, dynamic tire force measurements are unsatisfactory.

A measurement that is sensitive to the behavior of the vehicle on the highway should be used as a pavement condition criterion. Actually the dynamic tire force is a criterion of a combination of factors that includes pavement condition. Forces that are exerted against the pavement can be influenced by the vehicle suspension system and the speed of the vehicle. Inasmuch as these forces can be decreased, if necessary, by modifying the vehicle and by changing operating conditions, different vehicles, moving with different velocities, will each respond in a different manner to the same highway. The dynamic tire force is a measure of this response and for this purpose it is a valuable criterion.

It should also be pointed out that it is theoretically possible to remove both the vehicle characteristics and the vehicle veolcity from the tire force records, leaving a pavement characteristic that is free of these factors. This procedure is now receiving serious consideration.

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Addendum

The question has been raised as to whether or not it is possible to show a relationship between the values of dynamic tire force shown in Figure 7, and an objective rating of the associated pavement sections.

Fortunately, since the paper was written, it has been possible to obtain BPR roughometer ratings of the pavement sections under consideration.

In Figure 13 the logarithm of the dynamic tire force is plotted against the logarithm of the BPR roughometer rating for the pavement sections shown in Figure 7. Two highway sections have been omitted from this plot due to the lack of valid data. Each type of pavement has been indicated by a special symbol, and it is evident that the number of points for each type of pavement is very small.

Forming conclusions from the small quantity of data shown in the plot is hazardous, but certain trends may be suggested. In general, a large BPR roughometer rating is associated with a large dynamic tire force. Within limits, it may be possible to represent this relationship with a straight line as shown. If this is so, then a simple relationship can be used to predict the dynamic tire force from the BPR roughometer rating or vice versa. Vehicle parameters influencing the dynamic force (such as vehicle velocity, etc.) must not be changed if a relationship between dynamic force and roughometer rating is to be used. In addition, the curve shown in Figure 13 could not be used for heavy vehicles, because it was obtained by using a passenger car. It does suggest, however, that similar relationships may exist for different classes of vehicles. If this is true, then an estimate of the dynamic tire force to which a pavement would be subjected could be determined from the BPR roughometer rating of the pavement together with an appropriate curve of the type shown in Figure 13.

The question arises as to the effects of pavement construction (rigid, flexible or overlay) on the relationship between force and roughometer rating. Unfortunately there is not enough evidence to answer this question, but from the data shown in Figure 13 such an effect would appear to be of a minor nature.

The tests to date indicate that there is a relationship between dynamic tire force and BPR roughometer rating when the vehicle parameters are held constant. However, although a pavement will have only one roughometer rating, it will have different dynamic force values if the vehicle parameters are changed between successive tests of the same pavement.

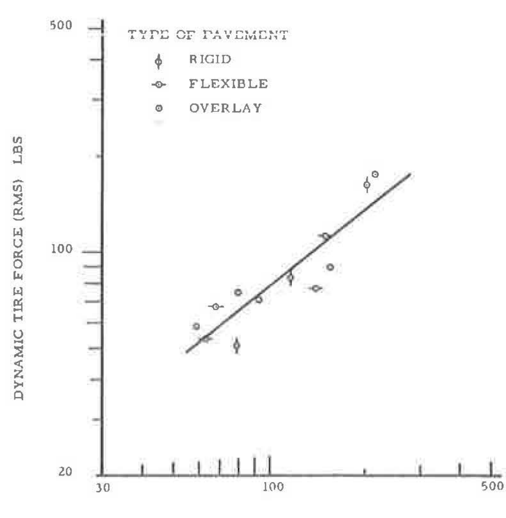


Figure 13. Relationship between dynamic tire force and BPR roughometer rating.

Portland Cement Concrete Airport Pavement Performance in Canada

G. Y. SEBASTYAN, Head, Engineering Design Section, Construction Branch, Canadian Department of Transport, Ottawa

In the first part of the paper portland cement concrete design considerations and construction practices used by the Canadian Department of Transport are discussed. This gives the background to the main subject of the paper showing the Department of Transport pavement evaluation procedures and experimental data for (a) portland cement concrete pavement strength, measured by field plate load tests; (b) curling of portland cement concrete pavements due to variation of the temperature gradient within the pavement; (c) the effect of curling of portland cement concrete pavement on pavement roughness; and (d) performance of airport portland cement concrete pavements in Canada.

• THE Canadian Department of Transport, Construction Branch, is responsible for the design and construction of all the major and most of the minor airports in Canada. There are 272 licensed and 481 unlicensed civil airports in the country, of which the Department owns and operates 117 and participates to varying degrees in the construction of the remainder.

DESIGN CONSIDERATIONS

Pavement design is based on static loading condition. Loadings are arranged in classes A to I, depending on airport class and operation.

Type A loading is 500-kip gross weight, 275-psi tire pressure (DC-8 wheel configuration). Type I loading is 27-kip gross weight, 50-psi tire pressure (single wheel).

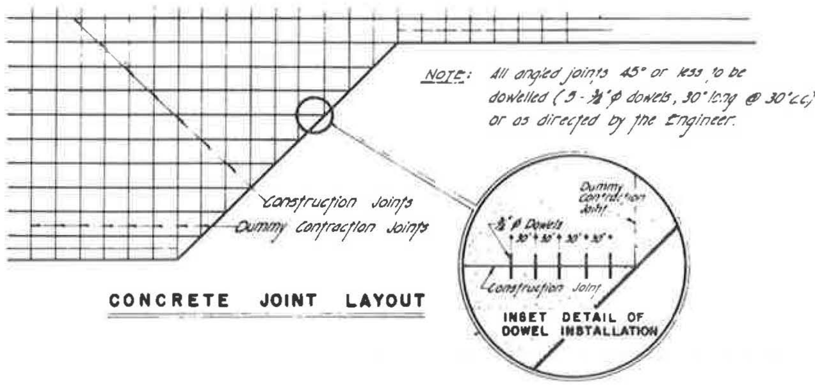
Under ordinary conditions the design policy of the Department is to construct aircraft parking aprons and runway ends in portland cement concrete. For the rest of the paved areas the choice of pavement surfacing material is made on the basis of economy.

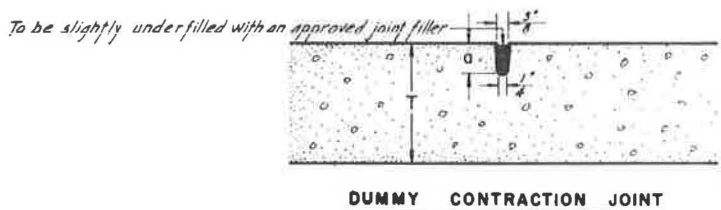
The standard design (1) is non-reinforced portland cement concrete. Slab sizes are 20 ft by 20 ft with a reduction for the edge slab to 12.5 ft by 20 ft. Expansion joints are not provided and no load transfer devices are used in the dummy joints. The construction joints are keyed. Standard joint details are shown in Figures 1, 2 and 3.

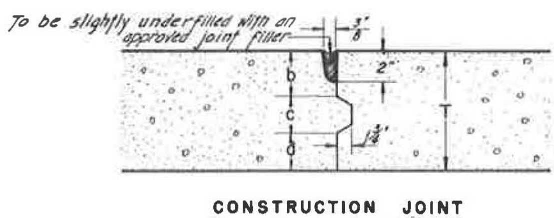
Pavement thickness is determined on the basis of the original Westergaard equation (2, 3), assuming a central loading condition. A safety factor of 1.2 is applied to the 28 days flexural strength of the concrete. At present, on the basis of field experience, it is considered advisable to limit the pavement thickness of portland cement concrete to 15 in. This limit is not usually exceeded even when the theoretical analysis would indicate otherwise.

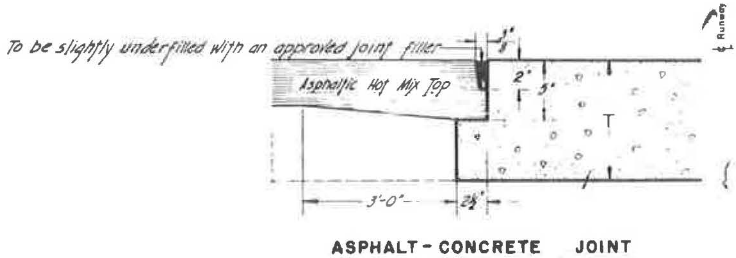
Protection against subgrade frost action is provided for in the design by the combined thickness of portland cement concrete pavement, base and subbase up to about one-half the expected depth of frost penetration, based on the 10-yr average freezing index, and the correlation presented in Figure 4.

Paper sponsored by Committee on Pavement Condition Evaluation.









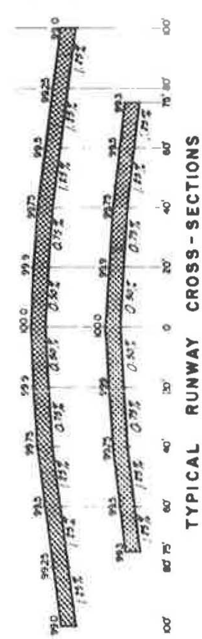
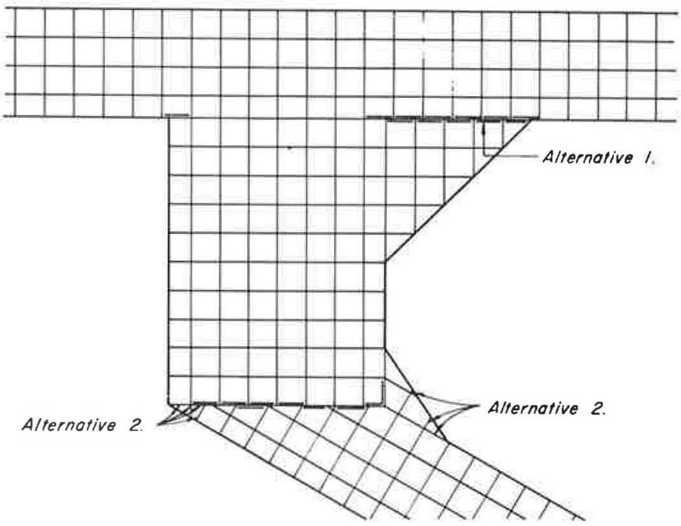


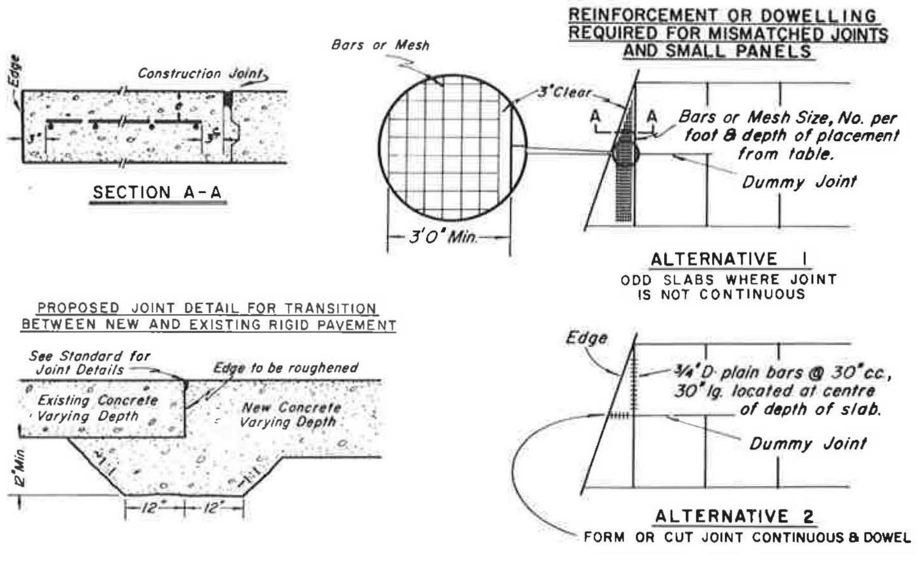
TABLE showing Dimensions for Keyed Construction Joints

T	a	b	c	d
8"	2"	3"	2½"	2½"
9"	2"	3½"	3"	2½"
10"	2"	4"	3"	3"
11"	2¼"	4"	3½	3½"
12"	2¼	4½"	4"	3½"
13"	2¼"	5"	4"	4"
14"	2½"	5½"	4½"	4"
15"	2¾"	5½"	5"	4½"
16"	3"	6"	5"	5"

Figure 1. Concrete joints' construction details.



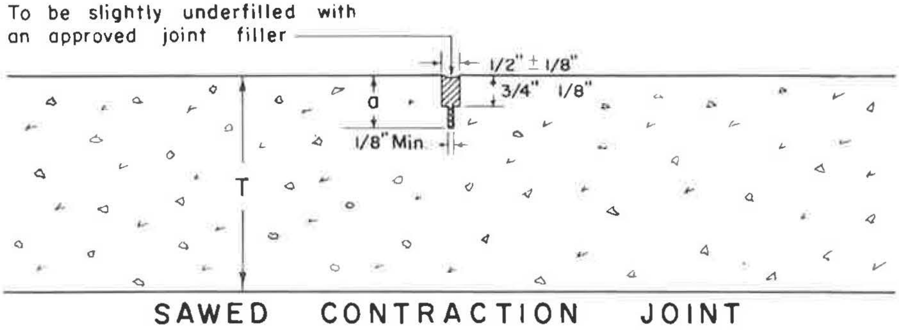
SUGGESTED LOCATION FOR USE OF STANDARD REINFORCING JOINTS



TABLE

SLAB DEPTH INCHES	BAR or MESH	NO. PER FOOT	DEPTH OF BAR
		Both Directions	e
9"	3/16"	2	3¼"
10"	5/16"	1	3½"
11"	5/16"	1	3¾"
12"	5/16"	1	4"
13"	¼"	2	4¼"
14"	¼"	2	4½"
15"	¼"	2	4¾"
16"	¼"	2	5"

Figure 2. Concrete joints' construction details.



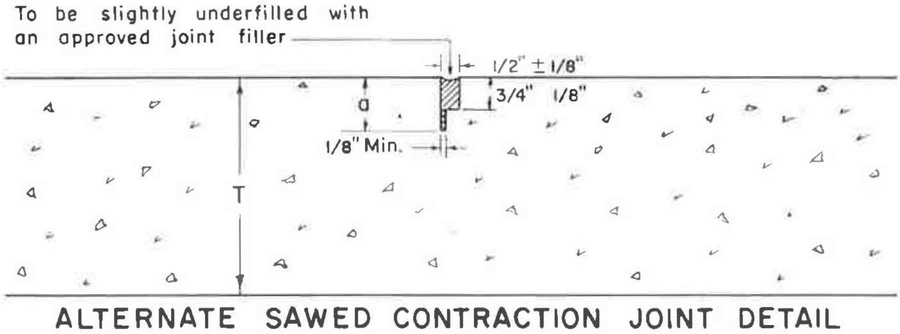


TABLE SHOWING JOINT DIMENSIONS				
T	a			
6"	1 1/4"			
7"	1 1/2"			
8"	2"			
9"	2"			
10"	2"			
11"	2 1/4"			
12"	2 1/4"			
13"	2 1/2"			
14"	2 1/2"			
15"	2 3/4"			
16"	3"			

Figure 3. Concrete joints' sawed joint details.

CONSTRUCTION PRACTICE

Paving operations are performed with concrete having as low slump as practical (close to zero slump) depending on the contractor's construction methods and machinery. Cement content varies between 5.5 to 6.0 Canadian bags of cement (87.5 lb) per cubic yard of concrete. This may be increased if conditions so dictate. Four to 6 percent of

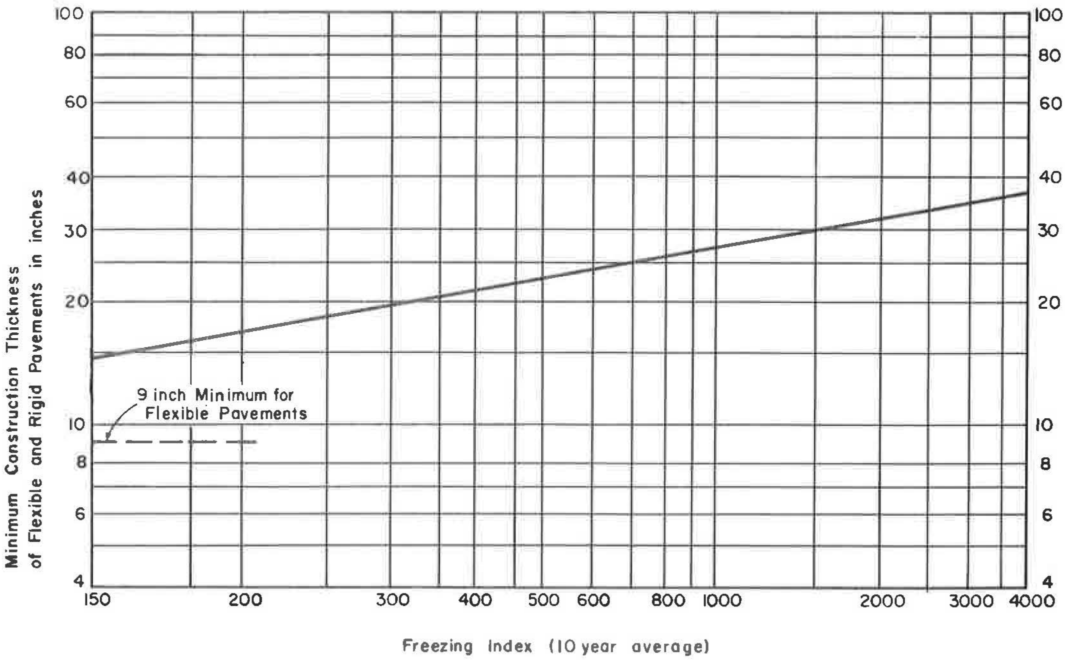


Figure 4. Minimum depth of frost protection for flexible and rigid pavements.

entrained air is obtained by use of an approved air-entraining agent. Except for this, further additives are used only in exceptional circumstances.

The quality of curing of the finished pavement affects strengths. One of the simplest methods of curing is the application of membrane curing compound. This is the method that most contractors choose from the alternatives specified in the Department of Transport Standard Specifications.

Difficulties were experienced with the quality of various membrane curing compounds supplied and also with the reproducibility of the quality control test results. During the winter of 1962, a program was set up by the Construction Branch of the Department of Transport and performed by the Department of Public Works Testing Laboratories to determine the major factors affecting moisture loss from the finished concrete after the application of concrete curing compound. The result of this testing program is given in the Appendix.

On the basis of the data obtained, the Standard Specifications have been changed to insure minimum acceptable solids content of the curing compounds and the reproducibility of the laboratory quality control test results.

Performance is related to built-in smoothness or roughness of the rigid pavement. It is affected by the quality of the joint forming operation, the placement and condition of the concrete forms used, the type of joint filling operation. A large variety of joint filling methods and filler materials are being laboratory and field tested to improve present practice.

Maximum deviation for irregularities of the finished pavement surface is specified in the Standard Specifications as ¼ in. in 15 ft.

PORTLAND CEMENT CONCRETE PAVEMENT STRENGTH AS MEASURED BY PLATE LOAD TESTING

As part of the Department's 1959/60 load testing program the static load-carrying capacity of portland cement concrete pavements was determined at five airport sites with varying subgrade soil conditions, subbases and portland cement concrete slab thicknesses (4).

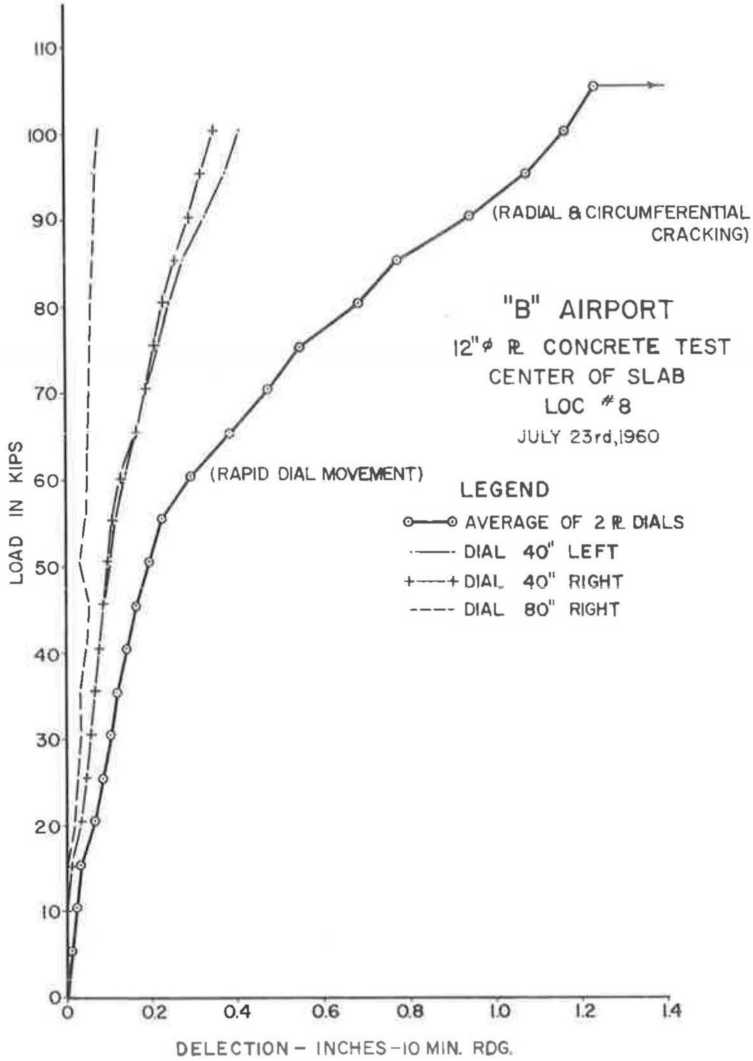


Figure 5. Sample load vs deflection graph.

Typical load-deflection diagram obtained from these tests is shown in Figure 5. As can be seen, the yield load (about 55 kips) and the collapse load (about 90 kips) can be differentiated.

Typical cracking pattern is shown in Figure 6. Radial and circumferential cracks are in evidence, which is in accordance with the plastic theory of plates. The field load-carrying capacity values obtained were compared by the theoretical load-carrying capacity computed on the basis of the Westergaard equation.

Figure 7 shows the ratio of static load-carrying capacity to theoretical strength (Westergaard analysis) in function of pavement thickness for center loading condition, for the free corner and for the protected corner case.

Similar comparison has been made by Meyerhof (8, 9) on the basis of an ultimate strength analysis using the plastic theory.

The data show that under static loading conditions, the Westergaard equation reproduced the load-carrying capacity fairly well for the free corner case. For the central loading conditions the static loads carried by the pavements have been considerably higher than predicted by the Westergaard equation.

On the basis of field performance of portland cement concrete pavements, it is the Department's experience that the Westergaard equation gives a conservative estimate of load-carrying capacity under Canadian construction, climatic and traffic environmental conditions.

CURLING

The absolute magnitude of portland cement concrete slab curling as a function of the temperature gradient within the slab was determined by an instrumented portland cement concrete slab at Halifax International Airport (1960 studies). Temperature instrumentation was provided by the Nova Scotia Technical College.

Details of the layout and installation of the measuring device are given in Figure 8. An example of the temperature regime of the 12-in. portland cement concrete slab for a 24-hr period (August 15-16, 1961) is shown in Figure 9. The temperature difference between the top and bottom of the slab for the same period is shown in Figure 10 and the relative movement of the slab under the influence of the given temperature gradient is shown in Figure 11. The maximum temperature difference between the top and bottom of the portland cement concrete slab for the period

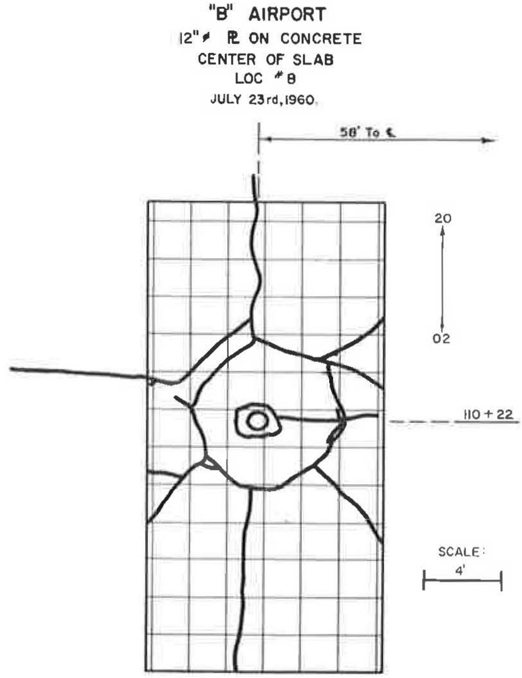


Figure 6. Sample slab cracking diagram.

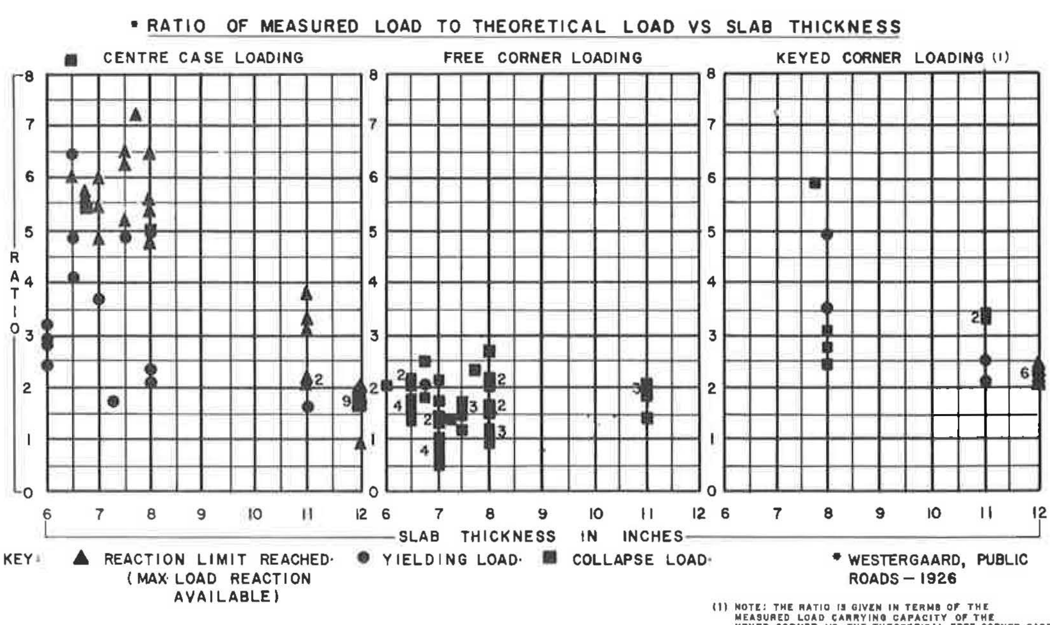


Figure 7. Load tests carried out on portland cement concrete airport pavements.

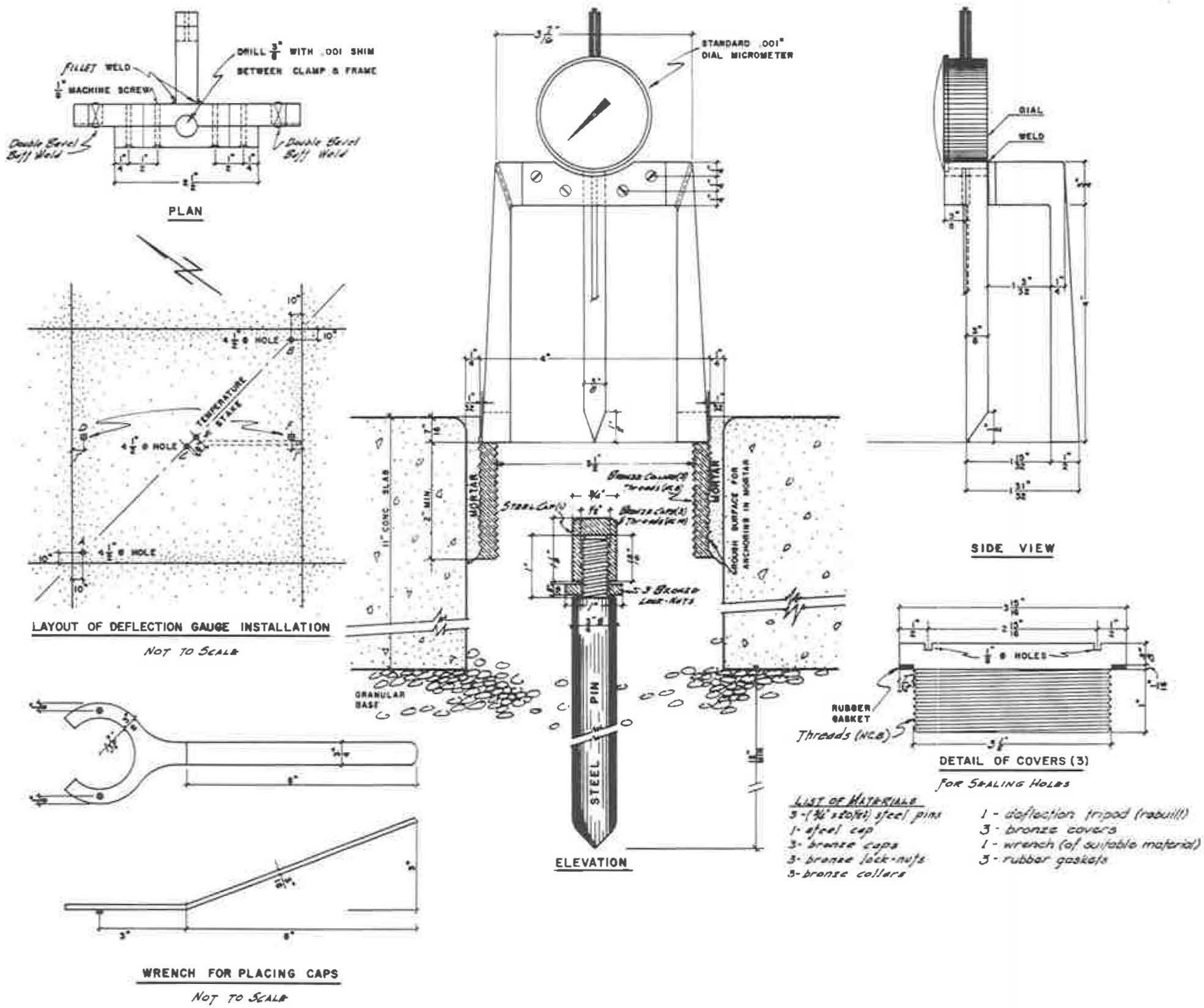


Figure 8. Proposed deflection gage installation.

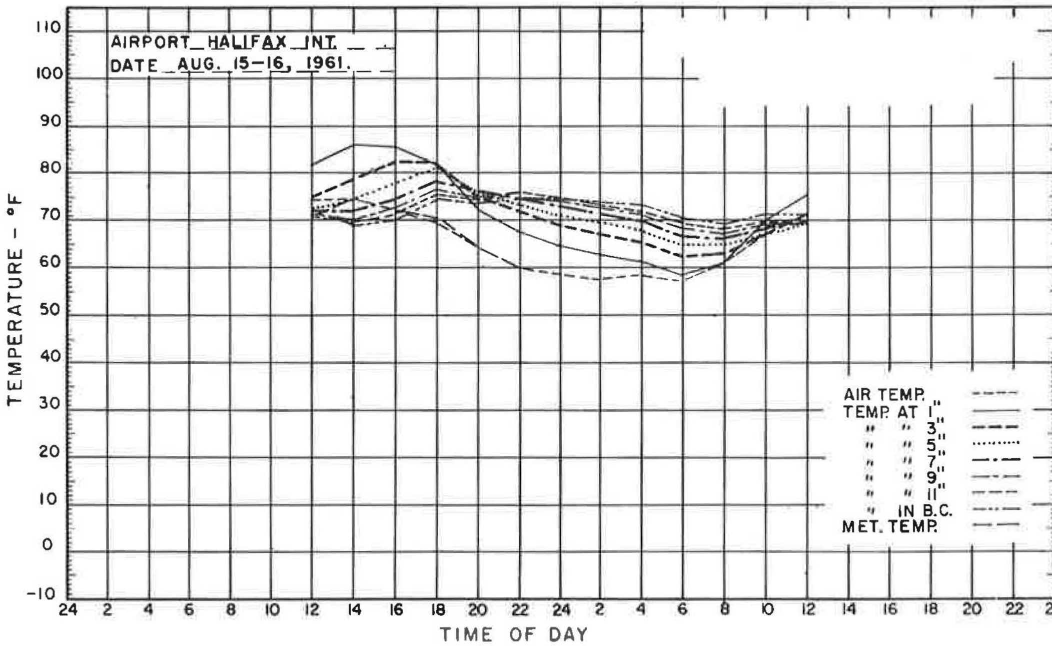


Figure 9. Portland cement concrete curling data: temperature vs time.

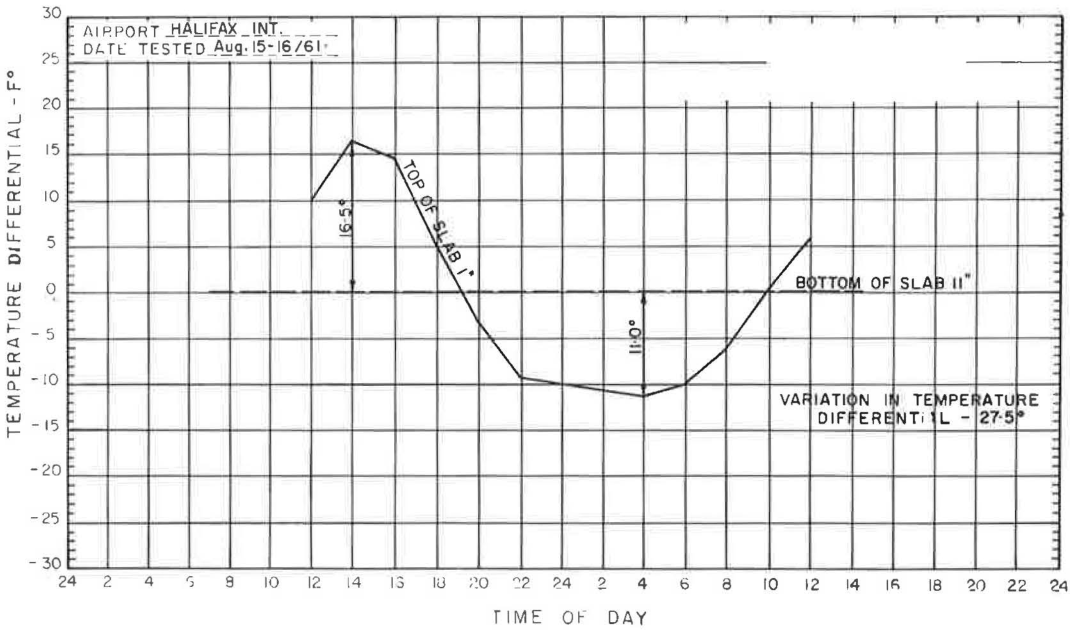


Figure 10. Portland cement concrete curling data: temperature differential vs time.

of one year (1961) is shown in Figure 12. The absolute magnitude of portland cement concrete slab curling (Department of Transport construction procedure) in function of the temperature gradient within the slab is shown in Figure 13.

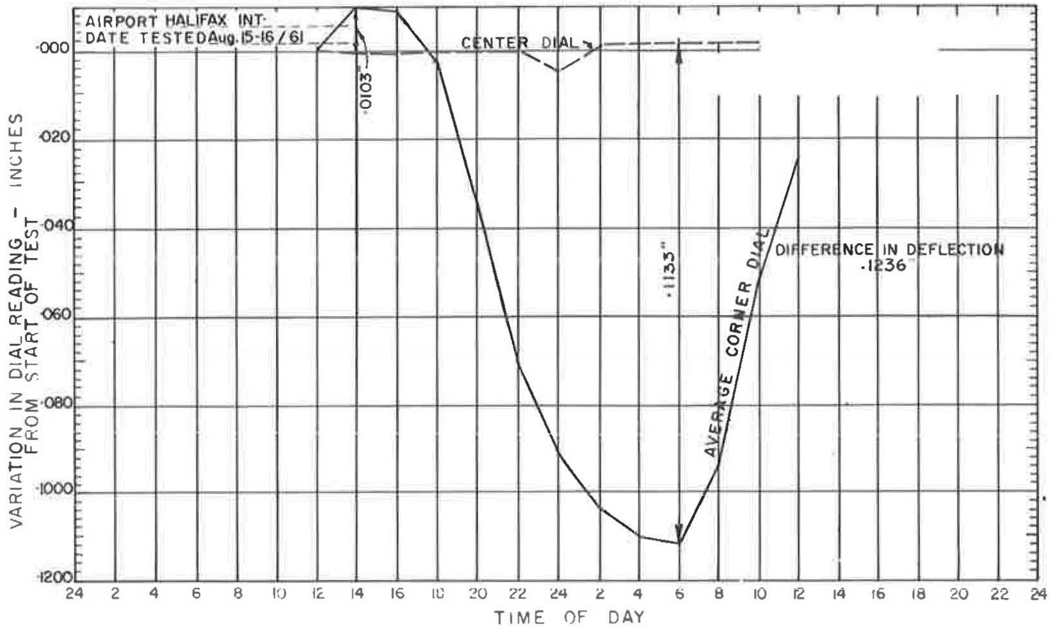


Figure 11. Portland cement concrete curling data: slab deflection vs time.

The maximum movement of the slab corner in respect to the middle of the slab was measured as more than 0.12 in. During static portland cement concrete load testing, initial cracking was observed at deflections slightly more than 0.1 in.

It was observed that in most of the cases maximum downward curling occurred about 2 PM and upward curling about 2-6 AM all year round. The maximum temperature difference within the concrete slab observed was 20 F at the end of July.

EFFECT OF CURLING OF PORTLAND CEMENT CONCRETE ON PAVEMENT ROUGHNESS

On the basis of the Halifax experiment the measured magnitude of curling was such that it influenced the smoothness of portland cement pavements.

To determine the influence of curling on the portland cement concrete roughness a program was initiated in 1962 and 1963 during which quantitative measurements were made on a given portland cement concrete pavement profile under varying temperature gradient conditions using the Department of Transport British-designed and built roughness measuring equipment (British Road Research Laboratories, 5).

A schematic sketch of the equipment is shown in Figure 14. The equipment provides a scale profile of the pavement surface using as a datum a floating level established by 16 irregularly spaced wheels; graphically integrates all the upward movements of the recording wheel, due to pavement irregularities relative to the floating level; and determines the distribution of the pavement roughness in 0.1-in. increments from 0.1 to 1.5 in.

For the purpose of this study the integration value (inch/mile) was used as a measure of roughness.

Two sets of results were obtained. One for a regular surface (Fig. 15) and another set for the same reference line, but the portland cement concrete surface was whitewashed. This, of course, changed the temperature regime within the slab (Fig. 16).

Figure 15 demonstrates that there is a definite variation in roughness during a given day under a given set of temperature gradients in the slab from the minimum value of 142 in./mi to the maximum of 167 in./mi measured under given conditions (17.6% increase).

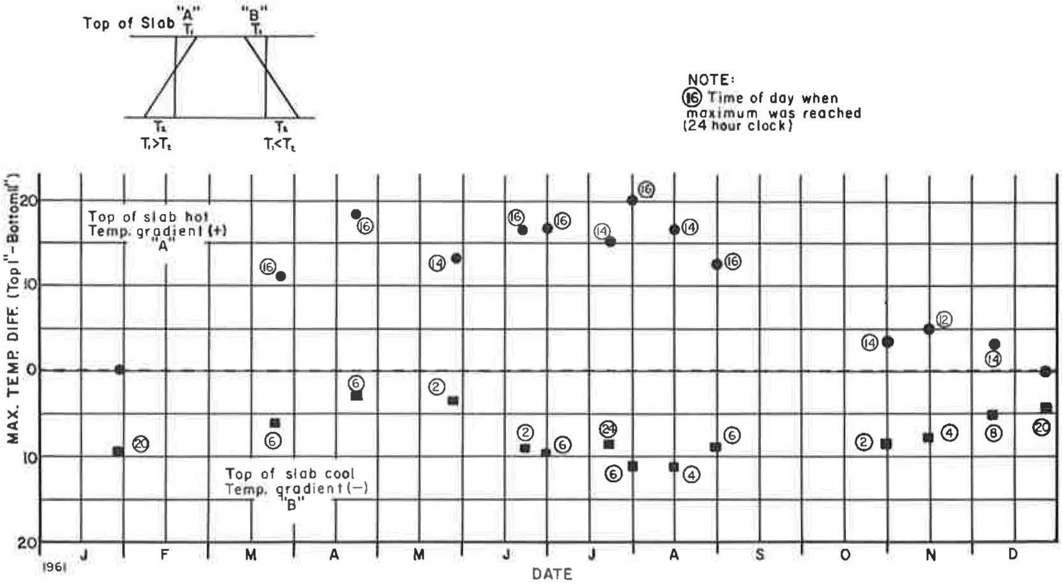


Figure 12. Maximum temperature difference vs time (top and bottom of portland cement concrete slab).

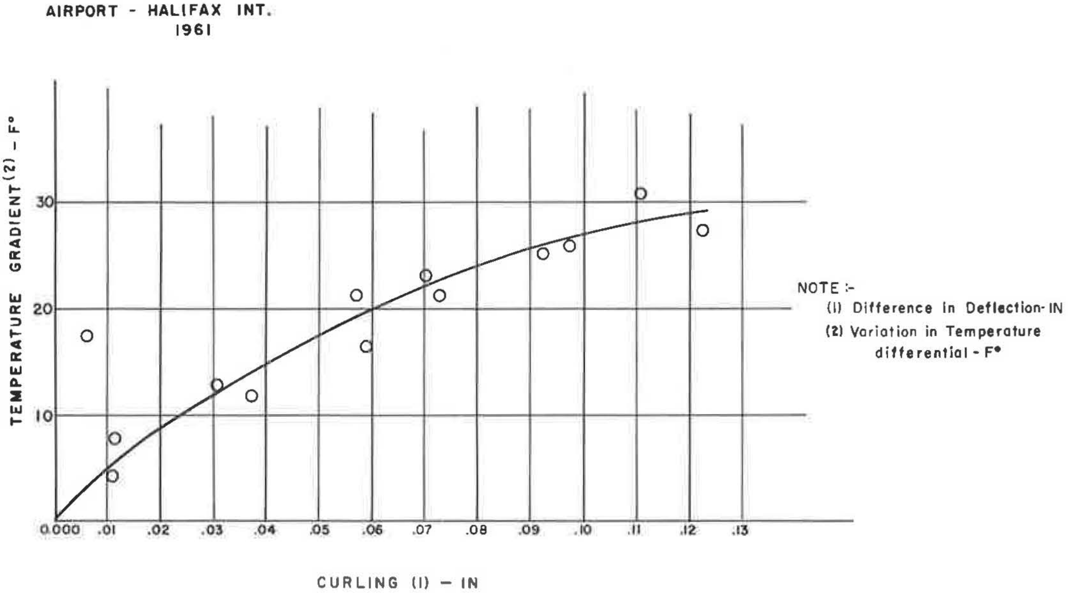


Figure 13. Portland cement concrete slab, curling results; temperature gradient vs curling.

Figure 16 shows that the whitewashed surface reduces the temperature gradient within the slab and consequently the resulting curling and roughness to the maximum and minimum values of 140 in./mi and 150 in./mi, respectively, under similar test conditions (7.1% increase).

Interpreting these results it is emphasized that the maximum air temperature difference was not identical in both measurements. (ΔT_1 = 32 F for the unpainted slab and ΔT_2 = 24 F for the whitewashed pavement.) The previous air temperature history will also influence the results.

PORTLAND CEMENT CONCRETE PAVEMENT PERFORMANCE

The determination of pavement performance is an integral part of the Department of Transport's pavement design and evaluation procedure.

Pavement performance is evaluated by the following factors: (a) pavement condition survey, and (b) pavement roughness measurements.

By performing pavement condition surveys, the structural continuity of the pavement surface is determined together with the possible causes of the various surface defects. The performance of such a survey is standardized in the Department of Transport Pavement Design and Construction Manual, Section 6, "Pavement Condition Survey."

Such a survey is always performed by experienced construction engineers. An example of the results of a portland cement concrete pavement condition survey for a given airport site is shown in Figure 17.

On the basis of these studies of 71 airport sites and the evaluation of 266 pavement units, the performance of Canadian portland cement concrete pavements has been summarized in function of pavement age in Figure 18.

Of course, there is considerable scatter of the data as a wide variety of subgrade types (soil ranges from GW to CH), portland cement concrete pavement thickness (from 7 to 14 in.), and environmental conditions (traffic density and intensity, freezing indices ranging from 800 to 5,000, etc.), is included in the summary.

As one of the results of the pavement condition survey, Figure 18 has considerable usefulness in planning. It is conditioned by the fact that aircraft traffic density and loading underwent revolutionary changes during the last 20 years, the time span of the service life of the pavements surveyed. Consequently, the findings are valid only for the Canadian Department of Transport Pavement Inventory.

A straight-line correlation was used between pavement performance and pavement age as the wide distribution of the data did not warrant the use of more complex function.

The data collected and the correlation presented might be used for the following:

1. Determination of the rate of depreciation in terms of time and cost of the Canadian airport pavement inventory as a whole (about 0.2 units per year — 2%).
2. On the basis of Department of Transport experience, Canadian airport construction practice, aircraft traffic and climatic environmental conditions, limited data indicate that major reconstruction of portland cement concrete pavements taking place in about 20 years at an approximate Department of Transport performance rating of 4.5.
3. Determination of the gained service life of the pavement by tighter quality control measures. For every 0.2 performance unit increase of the zero pavement age performance (as-built performance), the useful pavement life is extended by one year. On this basis, the value of quality control can be expressed in terms of direct monetary benefit.

Roughness measurements are made as part of the Department's pavement performance studies for the following reasons:

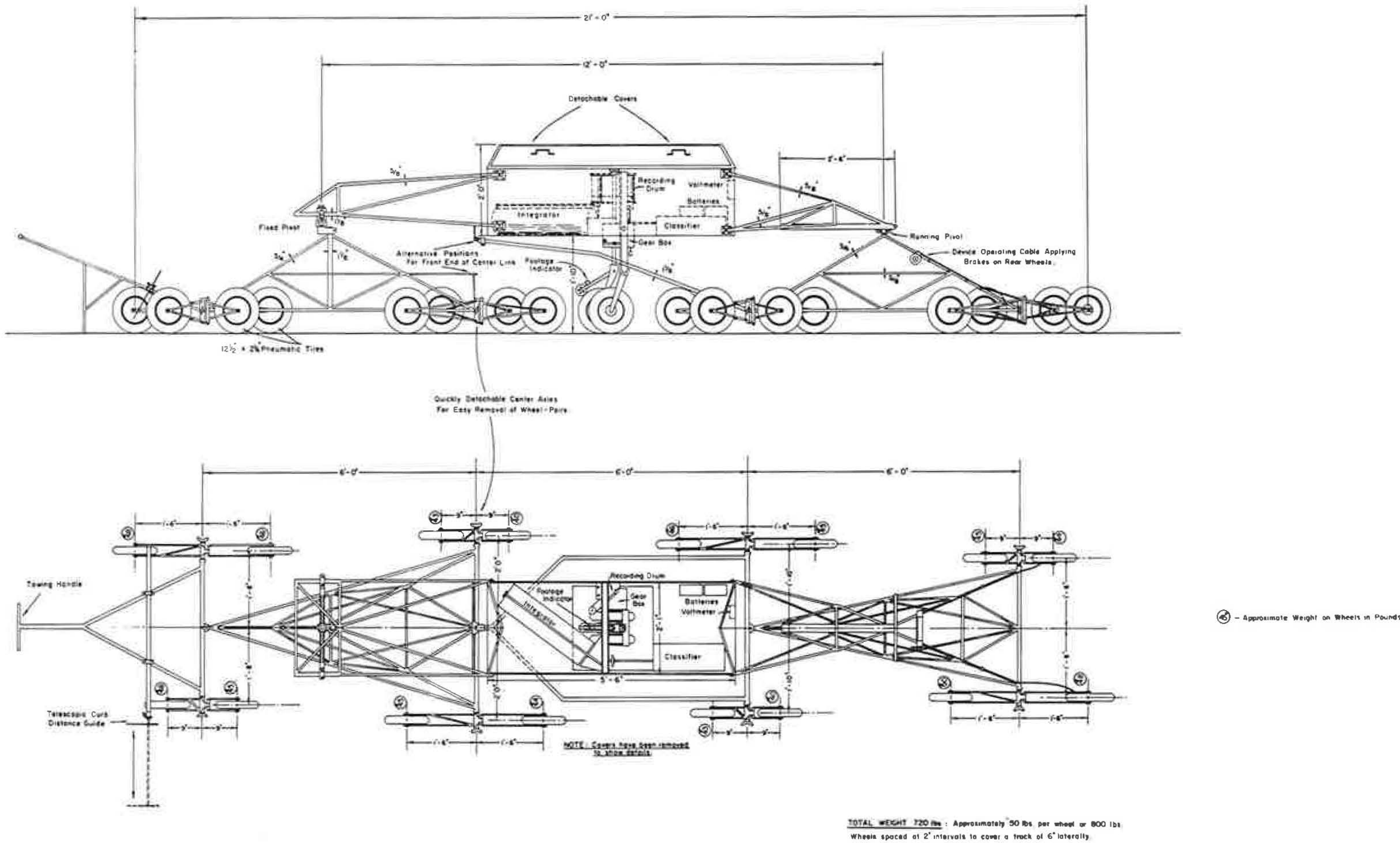


Figure 14. British Road Research Laboratory profilometer.

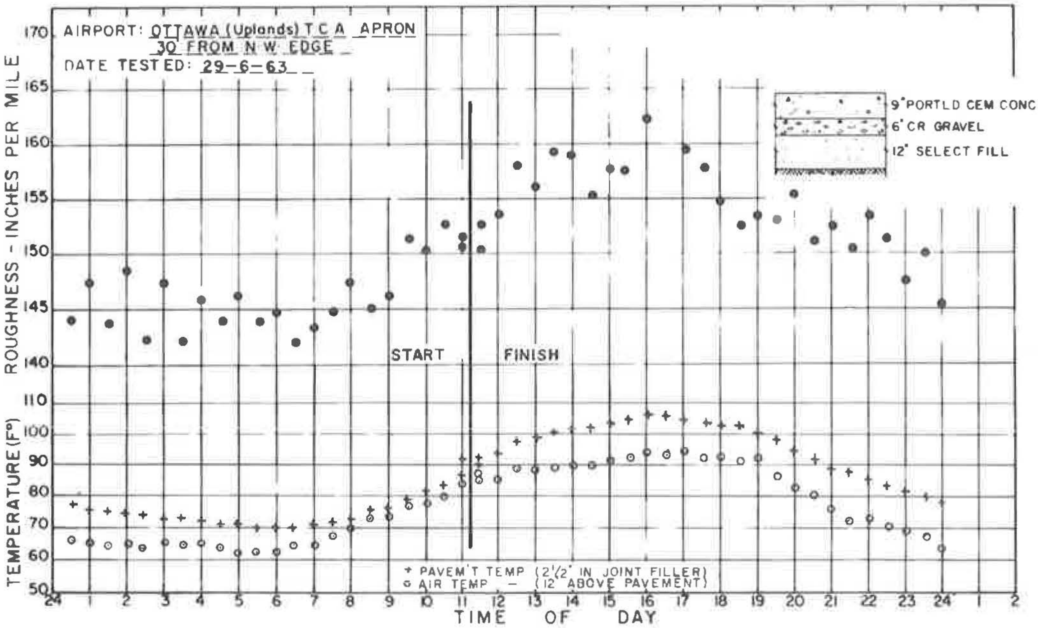


Figure 15. Plain portland cement concrete roughness tests: roughness and temperature vs time.

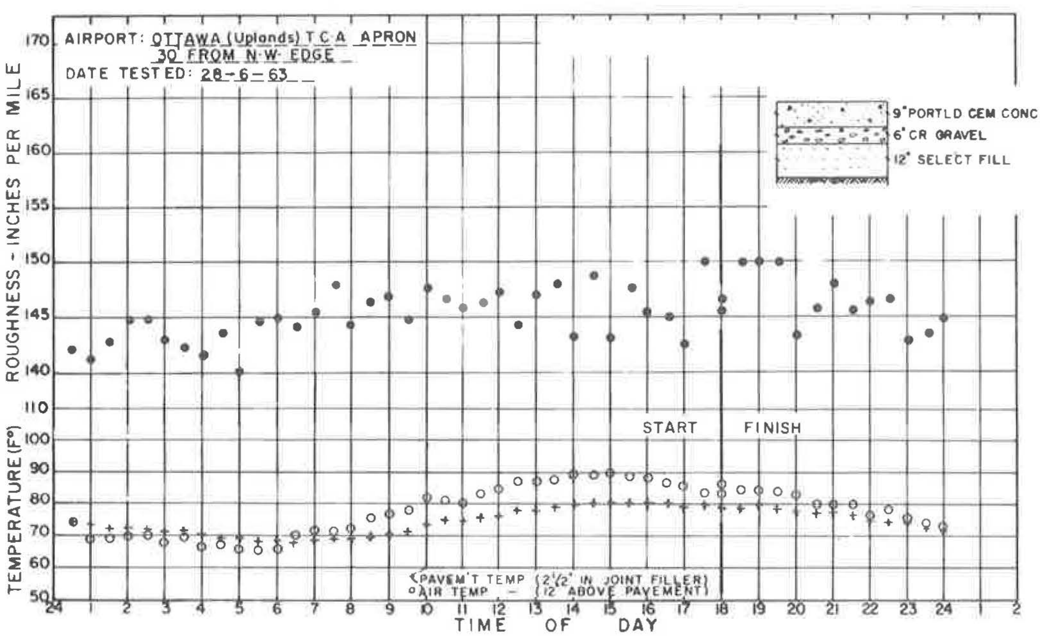


Figure 16. White portland cement concrete roughness tests: roughness and temperature vs time.

Revised 12/6/63

DEPARTMENT OF TRANSPORT

AIR SERVICES CONSTRUCTION BRANCH

CONDITION REPORT – RIGID PAVEMENT

AIRPORT AIRPORT "A"

OBSERVER J. FENNESSY

DATE 23 JULY 1963

EXTRA SHEET OF REMARKS 1

0: NONE
1: MINOR
2: MODERATE
3: MAJOR
4: SEVERE

10–9 VERY GOOD A
8–7 GOOD B
6–5 FAIR C
4–3 POOR D
2–1 VERY POOR E
0

		12 Button	30 Button	Elephant Ear At Old 25 Button	TAXI A	TAXI B	(Old) Apron I	(1955) Apron I	(1962) Apron I
CRACKING	CORNER	0	1[A]	2	2	2	2	1[E]	0
	EDGE	0	0	1	1	2	2	1[E]	0
	LONGITUDINAL	0	0	2	2[B]	3	2	0	0
	TRANSVERSE	0	0	3	3	3	3	0	0
	SCALING	0	0	1	1	1	2	0	0
	SPALLING	0	0	1	2	2	2	0	0
	JOINT STEPPING & FAULTING	0	0	2	2	1	2	0	0
	CONCRETE DISINTEGRATING	0	0	1	0	1	1	0	0
	PUMPING	0	0	2	0	0	0	0	0
	LOSS OF JOINT FILLING	0	0	1	1	0	1	0	2
	SUBGRADE SETTLEMENT	0	0	2	3	3	2	1[E]	0
	FROST HEAVE	0	0	1	1	1	1	1[E]	0
	PATCH	0	0	0	1	1	1	0	0
	LOCALIZED RECONSTRUCTION	0	0	0	1[D]	1[D]	1[D]	0	0
	SURFACE ROUGHNESS	7	7	5	4	4	4	6	5
	SURFACE DRAINAGE (PONDING)	8	8	7	8*	8	6	8	7
	SUBSURFACE DRAINAGE	7	7	7	7	7	7	7	-
	GENERAL CONDITION	9	9	5	5	4	5	7	8
		N	N	N	N	N	N	N	N
	WORK REQUIRED				C				

DRAINAGE REMARKS:

Drainage & Frost Heave Remarks & Ratings By Resident Engineer.

Graded Areas & Sides in Good Condition.

* Except Ponding shown as Major on Plan (TAXI A)

Most catch basins have heaved up.

Sub Surface Drainage good except often in Spring when open Ditch at East End of Field either freezes or plugs with snow, it causes water to back up & flood buildings and East End of the Field.

N = No sympathetic cracking

A = 1 only at Junction with flexible/Rigid Pavement

B = Almost entirely in outer bays (Centre 40' almost void of Longitudinal Cracks)

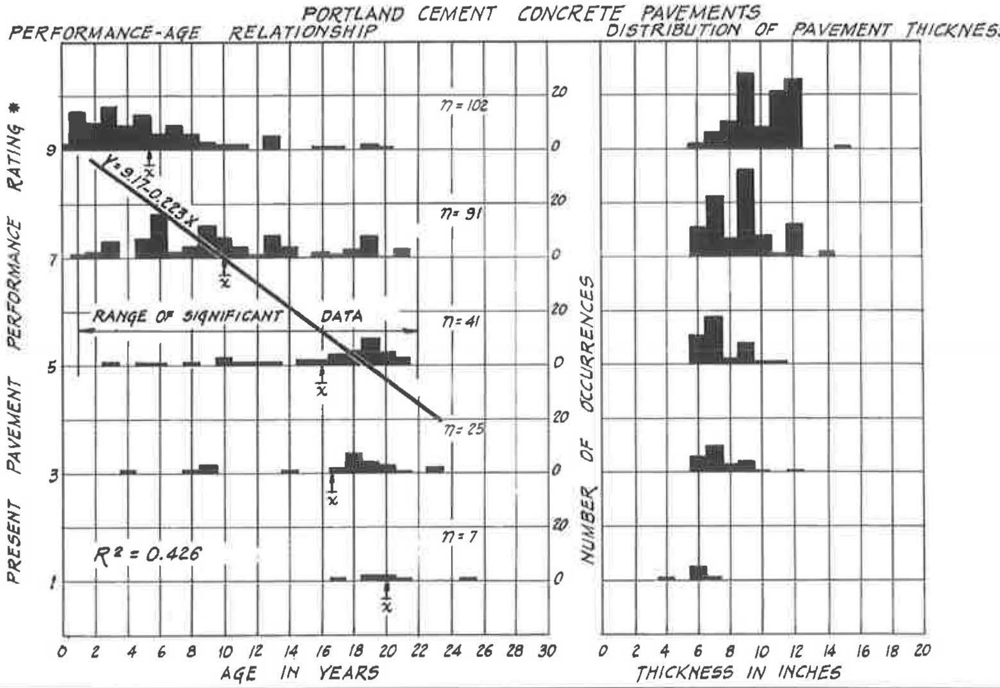
C = 75% cracking on Taxi A in outer Bays; caused probably by loss of Material under Edges.

GENERAL REMARKS:

D = Reconstructed after Ducts Laid.

E = Entirely at Raised Catch Basin near Taxi B.

Figure 17.



* Pavement Rating based on D.O.T. "Pavement Design & Construction Manual," Section 6, Pavement Condition Survey, March, 1963.
NOTE: 266 Sections investigated on 71 Airports

Figure 18.

1. Quality control check on new constructions.
2. Determination of the effect of environmental condition on pavement performance as expressed by roughness and change of roughness in function of time. Canadian Good Roads Association Special Technical Committee studies show that environment is one of the major influencing factors determining pavement performance. Both the AASHO Road Test and Canadian Good Roads Association studies show that pavement performance can be properly expressed in terms of pavement roughness.
3. Determination of the effect of pavement roughness on aircraft performance.

The roughness of pavements is measured in three phases:

1. Long wave roughness (over 25-ft wave length) measured by leveling.
2. Short wave roughness measured by the previously described profilometer.
3. Micro roughness (skid resistance) measured by the "Portable Skid Resistance Tester" developed by the Road Research Laboratories, England (6).

Typical short wave roughness index profile (inch per mile) is shown in Figure 19. The roughness index was based on measurements taken along the runway in the most probable wheel path of a DC-8 aircraft. Typical short wave roughness distribution diagrams are shown in Figure 20 based on roughness counter measurements (distribution of the size of pavement roughness in 0.1-in. increments).

In the Department of Transport experience the roughness index for a newly constructed portland cement concrete pavement with formed joints is about 60 in./mi. This value improves if the joints are sawn and the joint filling operation is properly performed. Roughness on in-service pavements was measured as high as 130 in. per mile.

Roughness measurements show that, in the Department of Transport construction practice, asphalt pavements are constructed considerably smoother than portland cement concrete pavements.

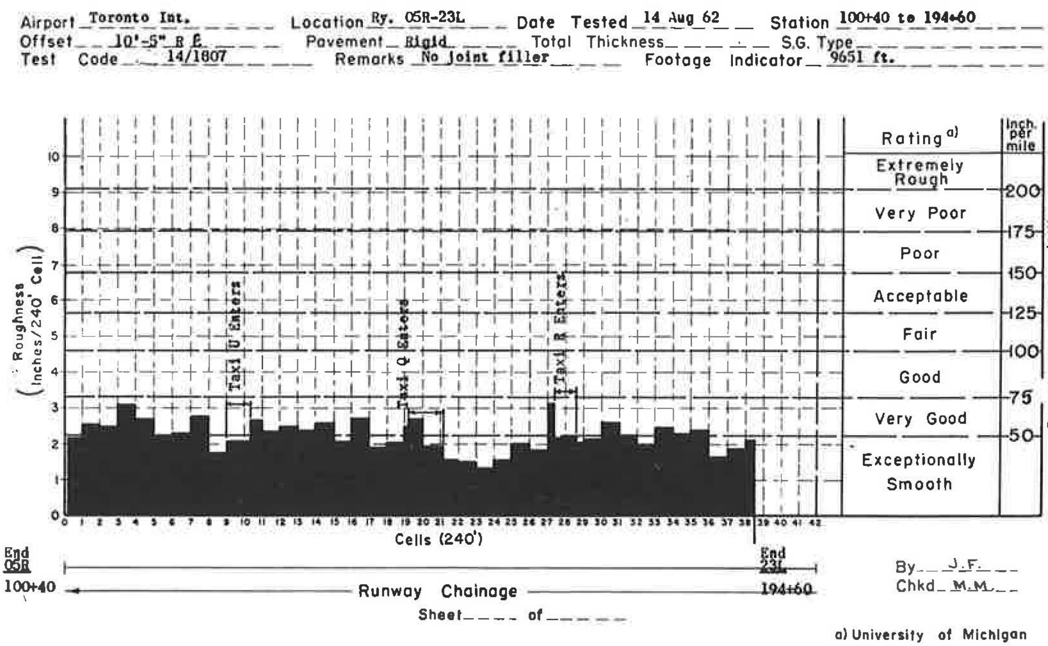


Figure 19. Profilometer data sheet (pavement roughness distribution).

TABLE 1

SUMMARY SHEET OF SKID RESISTANCE DATA

Airport	Date Tested	Pavement Type	100μ X	Range	No. of Tests
Sault Ste.	23 May '63	F	73.0	63-85	40
Marie		R	82.4	75-98	12
Sudbury	29 May '63	F	62.8	48-78	28
		R	70.6	59-79	14
North Bay	6 June '63	F	66.7	53-78	50
		R	73.4	67-82	24
Timmins	8 June '63	F	71.3	65-81	20
		R	—	—	—
Earlton	9 June '63	F	71.7	68-75	8
		R	—	—	—
Lakehead	22 July '63	F	72.1	66-80	18
		R	73.8	67-83	8
Winnipeg	27 July '63	F	73.3	62-83	33
		R	68.9	55-95	46
Portage la	14 Aug. '63	F	71.4	65-77	34
Prairie		R	68.6	55-78	10
Regina	16 Aug. '63	F	68.6	51-83	18
		R	73.4	65-80	14
Saskatoon	28 Aug. '63	F	80.7	71-87	28
		R	77.6	70-84	26
Cold Lake	6 Sept. '63	F	74.4	65-82	46
		R	72.0	60-83	20
Namao	20 Sept. '63	F	77.3	60-88	45
		R	72.5	59-63	34
Edmonton	3 Oct. '63	F	—	—	—
		R	73.5	65-82	86
Lethbridge	17 Oct. '63	F	79.9	74-85	25
		R	76.6	72-79	6
Calgary	25 Oct. '63	F	84.0	41-102	74
		R	81.6	63-96	13
Victoria	20 Nov. '63	F	82.2	70-89	47
		R	86.0	82-95	6

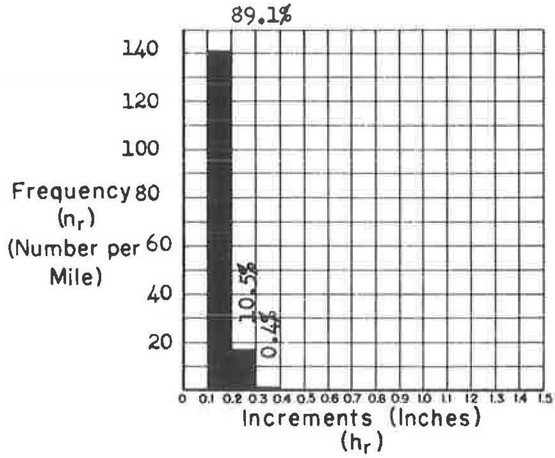
PROFILOMETER DATA SHEET
(FREQUENCY HISTOGRAM)

Airport Toronto Inter'l. Location Ry. 05R-23L Date Tested 14 Aug '62

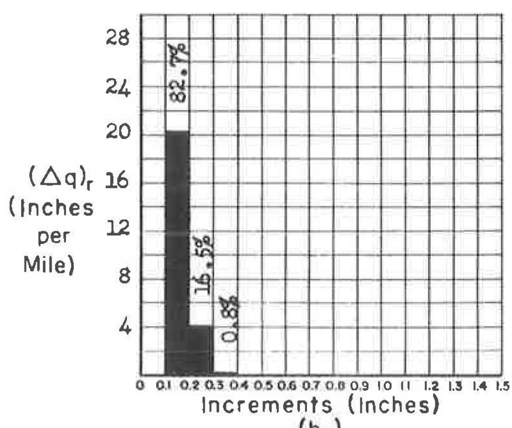
Station 100+40 - 194+60 Offset 10' 5" R of ℄

Pavement Rigid Total Thickness S.G. Type

Test Code: YZ 14/807 Remarks: No Joint Filler



h_r	n'_r	n_r	$\bar{h}_r$	$(\Delta q)_r$
.1	245	140.9	.145	20.43
.2	29	16.7	.245	4.09
.3	1	0.6	.330	.20
.4				
.5				
.6				
.7				
.8				
.9				
1.0				
1.1				
1.2				
1.3				
1.4				
1.5				
	Total		q =	24.72



$(\Delta q)_r = n_r \times \bar{h}_r$

$$\left[\begin{array}{l} \bar{h}_r = h_r + 0.045 \\ \bar{h}_{r_{MAX.}} = h_{r_{MAX.}} + 0.030 \end{array}\right]$$

n'_r = Number per test length (footage indicator)

$n_r = n'_r \times f$

$f = \frac{5280}{9190} = 0.575$

Footage Indicator 9651 ft.

Footage Correction Factor 1/1.05

Corrected Footage 9190 ft.

Δp = 85.19 inches

q = Classifier Index	24.7 inches/mile
p = Integrator Index	49.0 inches/mile

By J.F.

Chkd M.M.

Figure 20.

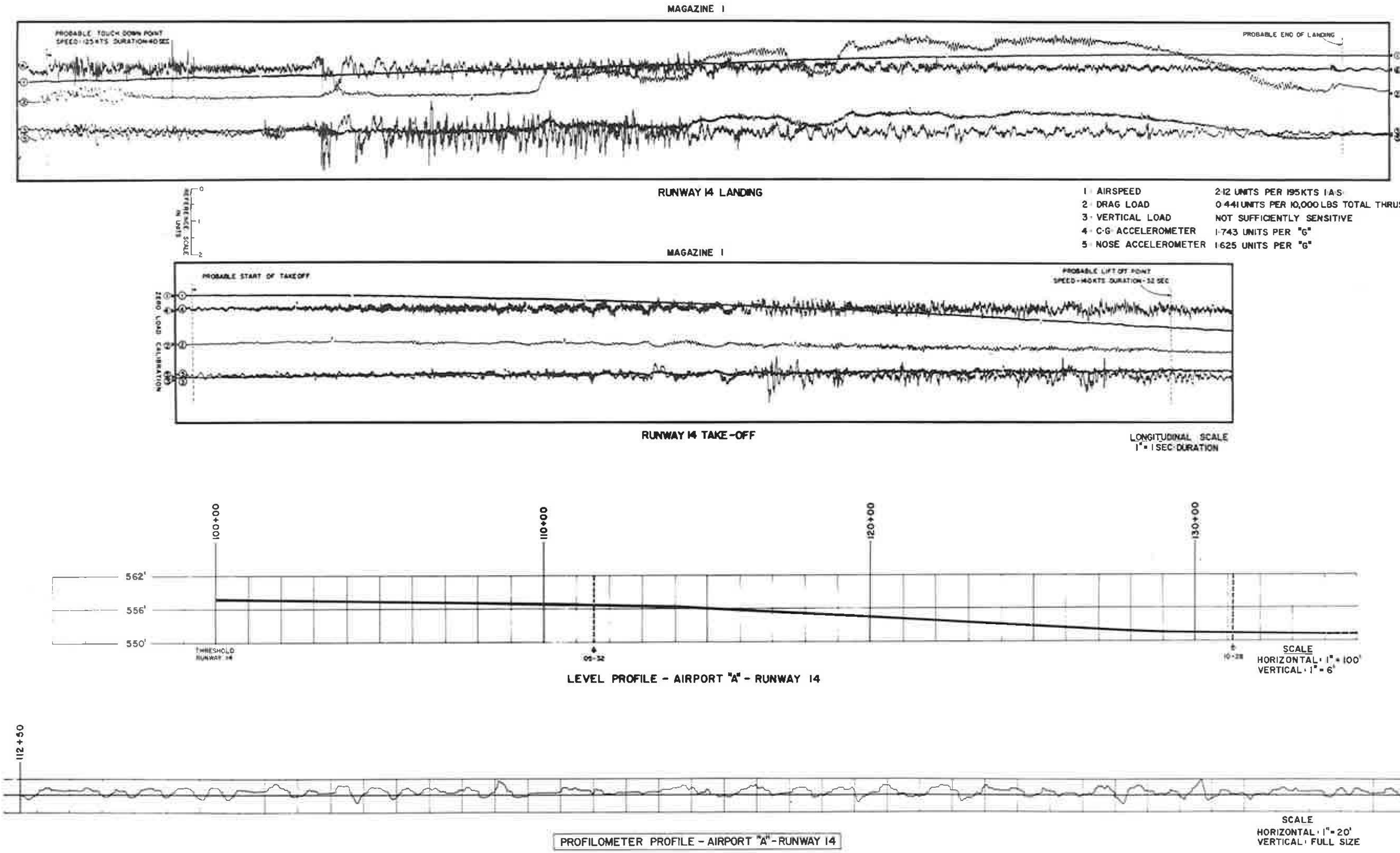


Figure 21. Typical pavement roughness data and dynamic aircraft (DC-8) response.

TABLE 2

SUMMARY OF DC-8 RESPONSE TO ROUGHNESS DATA (1963)

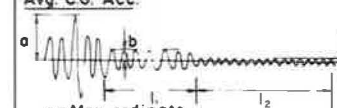
Airport	Op.	Runway	Air Speed lift off or touchdown	A/C Op. Weight	Max C.G. Acc. Ord.	Avg C.G. Acc. Ord.	Max Nose Acc. Ord.	Avg Nose Acc. Ord.	Max Horiz Ld. Leg[1]
			kts	kips	g	g	g	g	kips
UL	T.O.	06R	136	213·5	1·63	1·039			−4·4
	L.	06L	120	183·3	1·68	1·056	1·30	1·074	18·8
	T.O.	28	138	217·4	1·39	1·054	1·21	1·036	7·9
	L.	24L	110	192·0	1·29	1·069	1·65	1·090	18·4
	T.O.	28	138	201·0	1·21	1·054	1·23	1·049	−3·9
	L.	28	115	189·0	1·72	1·079	1·36	1·071	20·5
	T.O.	28	144	207·8	1·36	1·064	1·30	1·043	−4·2
	L.	24L	117	188·0	1·42	1·062	1·33	1·078	12·0
	T.O.	06R	138	210·3	1·20	1·049	1·18	1·043	−8·6
	T.O.	28	132	204·6			1·30	1·052	−9·9
	L.	24L	110	169·8			1·71	1·126	
	T.O.	24R	140	210·6			1·22	1·060	−6·5
	L.	06L	125	190·0			1·34	1·061	8·2
YZ	L.	14	125	197·7	1·52	1·064	1·63	1·095	20·3
	T.O.	14	140	238·6	1·21	1·049	1·36	1·045	−6·0
	L.	28	123	203·6	1·53	1·061	1·36	1·064	17·2
	T.O.	28	142	244·8	1·30	1·048	1·28	1·034	−7·0
	L.	14	112	176·0	1·63	1·036	1·41	1·096	28·3
	T.O.	32	132	205·9	1·64	1·051	1·28	1·027	4·2
	L.	14	124	185·5	1·67	1·084	1·84	1·087	20·4
	T.O.	28	142	242·3	1·18	1·040	1·18	1·045	−5·1
	L.	32	110	176·4	1·45	1·072	1·63	1·073	17·2
	T.O.	32	138	202·9	1·63	1·056	1·26	1·030	−5·8
	L.	32	123	193·3	1·57	1·096	1·44	1·093	12·8
YZ	T.O.	32	145	240·5	1·48	1·062	1·32	1·055	−4·1
	L.	32	122	193·4	1·48	1·082	1·62	1·096	11·0
	T.O.	32	140	203·0	1·21	1·043	1·34	1·035	−8·2
	T.O.	32	144	239·2	1·41	1·067	1·33	1·060	−12·9
	L.	32	118	199·8			1·34	1·068	−5·0
	L.	14	115	190·2			1·42	1·102	
	T.O.	14	125	226·1			1·26	1·078	
	L.	14	122	194·9			1·39	1·122	22·4
	T.O.	32	140	235·6			1·47	1·059	−10·9
	T.O.	32	139	204·9			1·22	1·050	−3·2

REMARKS

1. Largest numerical value.

Blanks indicate missing curves on film.

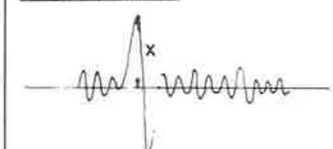
Avg. C.G. Acc.



a = Max ordinate

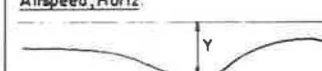
$$\text{Avg. C.G.} = \frac{bl_1 + cl_2}{l_1 + l_2}$$

Max. Acc. Reading



Max = 1+X

Airspeed, Horiz.



Max = Y

YC	L.	28	126	185·6	1·66	1·056	1·49	1·076	25·2
	T.O.	28	140	201·6	1·14	1·041	1·15	1·030	−2·9
	L.	28	130	185·6	1·40	1·052	1·49	1·064	30·0
	T.O.	28	152	232·9	1·16	1·038	1·23	1·038	−4·6
	L.	28	118	195·0	1·36	1·042	1·50	1·071	24·8
	T.O.	28	150	208·2	1·47	1·039	1·22	1·015	−5·1
	L.	28	120	200·0	1·22	1·050	1·33	1·061	16·5
	T.O.	28	134	224·5	1·36	1·045	1·17	1·018	−3·4
	L.	28	128	190·3	1·54	1·062	1·34	1·056	16·4
	T.O.	10	143	206·4	1·15	1·034	1·14	1·020	−2·6
	L.	10	118	186·1	1·40	1·051	1·53	1·059	−21·4
	T.O.	10	140	221·2	1·17	1·043	1·17	1·016	−3·7
	L.	10	121	185·5	1·28	1·036	1·31	1·040	19·9
	T.O.	10	143	194·4	1·14	1·038	1·15	1·029	−2·0
YC	L.	10	117	177·3	1·37	1·063	1·40	1·081	15·3
	T.O.	10	142	229·3	1·19	1·040	1·18	1·029	−2·7
	L.	28	131	189·4			1·37	1·086	20·8
	T.O.	28	136	195·1	1·14	1·035	1·16	1·033	−7·3
	L.	28	115	177·2			1·46	1·080	9·9
	T.O.	28	142	229·1	1·33	1·037	1·17	1·029	−7·7
VR	L.	08	124	184·6	1·36	1·050	1·74	1·053	11·8
	T.O.	08	148	203·9	1·26	1·046	1·18	1·034	−3·9
	L.	26	118	193·6	1·28	1·052	1·30	1·062	25·5
	T.O.	26	128	218·6	1·44	1·036	1·26	1·023	−6·3
	L.	26	125	191·1	1·50	1·043	1·36	1·059	9·4
	T.O.	26	141	205·5	1·17	1·043	1·14	1·027	−5·3
	L.	08	122	178·1	1·23	1·032	1·35	1·053	17·2
	T.O.	26	142	195·5	1·14	1·037	1·12	1·019	−2·6
	L.	08	128	178·8			1·45	1·088	10·4
	T.O.	08	140	193·6	1·17	1·045	1·16	1·030	−5·9
	L.	08	115	176·0			1·44	1·088	
	T.O.	08	138	192·4			1·36	1·066	
	L.	26	118	179·0			1·33	1·086	12·9
	T.O.	26	140	196·0			1·11	1·021	−6·5
XD	L.	11	128	173·7			1·88	1·111	
	T.O.	11	140	195·2			1·27	1·088	
	L.	11	110	174·3			1·52	1·162	
XD	T.O.	11	138	234·6			2·12	1·070	
	L.	19	116	182·3			1·80	1·095	8·8
	T.O.	19	140	196·9			1·20	1·056	−6·3
	L.	19	127	177·0			1·43	1·101	20·5
	T.O.	19	133	232·9			1·29	1·061	−6·9

Data evaluation and presentation are still in an experimental state. At present the data are evaluated on the inch-per-mile roughness index basis. Studies performed on the collected data show that the variance of roughness measurements is a better measure of pavement roughness than the inch-per-mile index value. This, of course, confirms the recent AASHO Road Test results.

A study is in progress to express roughness in terms of physical quantities which would allow a dynamic analysis. A statistical measure is being sought which would express roughness on the basis of wave length and amplitude frequencies obtained from the raw data without mathematical manipulation.

Micro roughness (skid resistance) measurements are given in Table 1 (not an absolute value as it is related to the measuring equipment used and the technique employed). Measurements are made on wet surfaces. No complaints have been received from pilots and operators to date. It is considered that the quoted order of magnitude of 0.65-0.80 is well within the limit of safe aircraft operation.

For the measurements of actual aircraft loading on pavements and aircraft response to pavement roughness, an in-service DC-8 aircraft was instrumented.

This project was carried out in cooperation with Trans-Canada Air Lines and the National Aeronautical Establishment of the National Research Council of Canada.

The following instrumentation was placed on the aircraft, in accordance with the recommendations of the Douglas Aircraft Company: (a) center of gravity acceleration, (b) acceleration of the nose wheel, and (c) main gear load (C-1 vertical, C-2 horizontal).

Typical ground roughness and aircraft response measurements are shown in Figure 21.

The data are in the process of analysis. The order of magnitude of some of the average and maximum results obtained during given operations is summarized in Table 2. The maximum horizontal load measured was 28.3 kips. Landing and take-off speeds and the average level of aircraft response to pavement roughness are also indicated. Work is in progress to establish a statistically significant measure of aircraft response to any given pavement roughness.

This will help to establish construction specification limits for new construction and to determine the necessity of major maintenance operation for in-service pavements.

The vertical strain gages located on the main under-carriage did not give significant results, because they were installed in a location where the vertical and horizontal strain components interacted.

The vertical load on the pavement was estimated on the basis of the "acceleration factor" measured in the center of gravity of the aircraft.

During the test operations the following maximum acceleration factors were measured: taxiing operation, 1.31 (104 operations); landing operation, 1.72 (52 operations); take-off operation, 1.64 (52 operations).

During landing and take-off operations, part of the aircraft weight is carried by the wings, depending on aircraft speed, braking action, thrust reversal, and the use of "spoilers" and "flaps."

Taking into account all these factors, it is estimated that the maximum load acting on the pavement is 1.5 times the aircraft gross weight under regular operating conditions.

In the Department's pavement design practice this impact factor is not taken into consideration as the subgrade soil is able to sustain high intensity loading of short duration without appreciable amount of deformation.

ACKNOWLEDGMENTS

The investigations presented in this paper were carried out under the administration of G. W. Smith, Director of Construction Branch.

The instrumentation for the temperature gradient in the concrete slabs was carried out by the Nova Scotia Technical College and the testing program was carried out under the direction of L. B. Hunter, Moncton Regional Materials Engineer.

The following engineers of the Engineering Design Section were responsible for the various programs: E. Penner (Load Testing), J. Demellweek, J. Fennessy, G. H. Argue (Roughness Surveys), and H. Dutz (Pavement Performance Studies).

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Appendix

OBSERVATIONS AND COMMENTS ON THE ASTM TEST AND SPECIFICATION FOR MEMBRANE FORMING CURING COMPOUNDS

In ASTM Standards C 156-55T and C 309-58, the type of brush used and the time of brushing of the surface of the sample has not been clearly defined. Laboratory test results shown in Figure 22 show the influence of the type of brush used and the time of brushing on the moisture loss. The relationship between the percent non-volatile solids on the moisture loss as function of time of curing compound application is shown in Figure 23.

The time of brushing of the surface of the samples and the type of brush used has influenced the formation of laitance. If such laitance is formed, the effectiveness of the concrete curing compound water retention capacity is reduced.

Also the time of application of the compound is critical. If the compound is applied when the surface has dried out to a critical degree, the concrete might absorb some of the applied material and pinholes could form in the surface making it possible for moisture to evaporate from the concrete.

For laboratory acceptance testing of concrete curing compounds, at present 2.5 hours are specified as maximum application time in Department of Transport specifications. This is a conservative estimate of field conditions.

To insure proper curing the percentage of non-volatile solids is of course of primary importance. The function presented in Figure 23 demonstrates this clearly. On the basis of 2.5-hr maximum application time and the correlations obtained in Figures 22 and 23, the minimum solid content of 30% is insured.

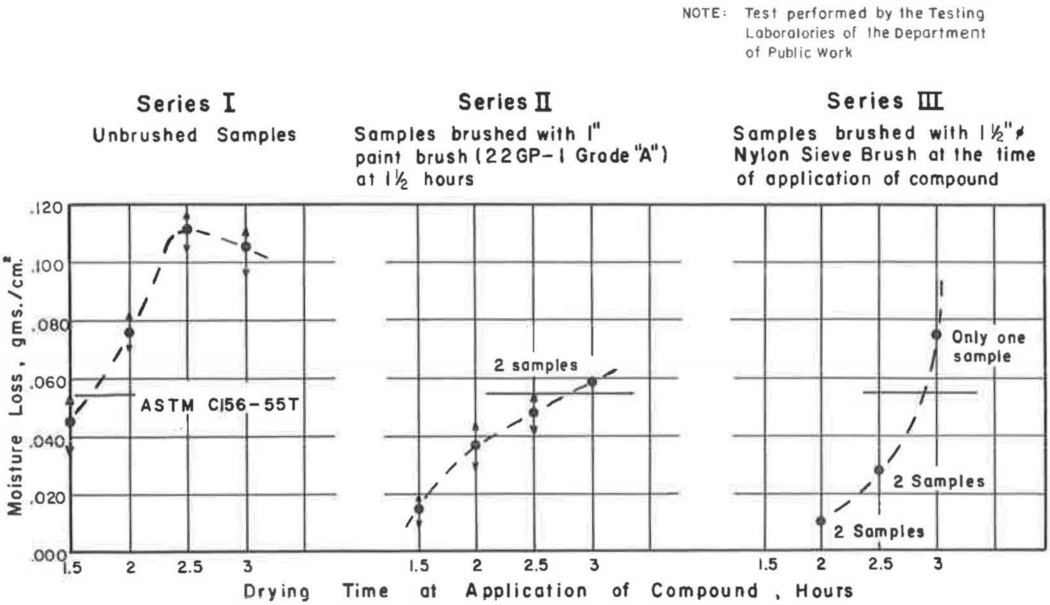

Figure 22. Concrete curing compounds: moisture loss vs drying time (30% non-volatile solids).

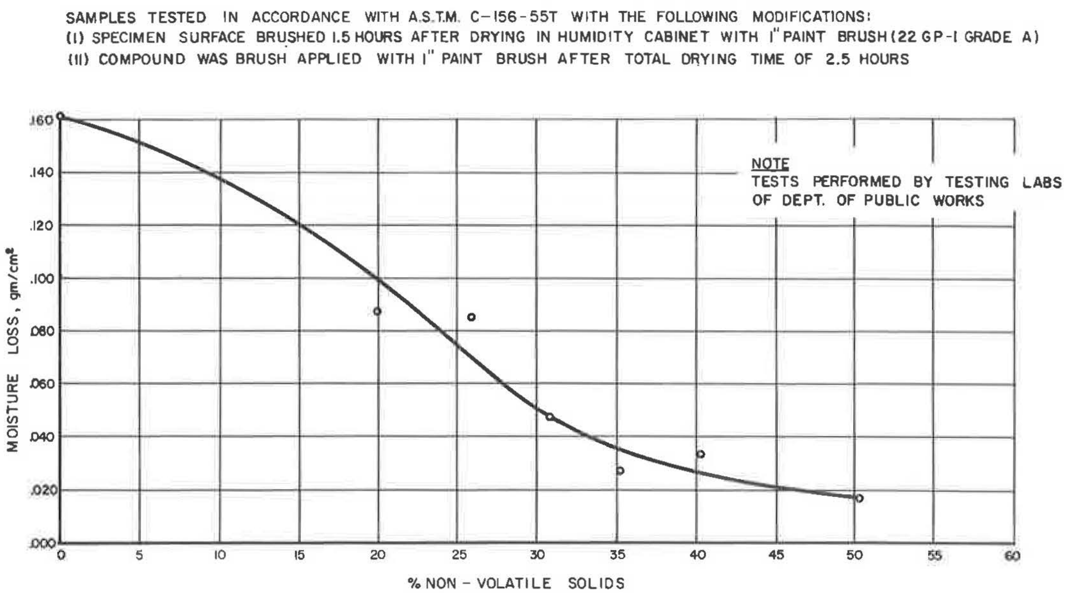

Figure 23. Concrete curing compound: moisture loss vs solid content.

Computation of True Highway Elevations from An Elevation Profilometer Record

G. RAGNAR INGIMARSSON
Assistant in Research, University of Michigan

This paper presents the mathematical development of and the procedure for the conversion of the modified pavement profile recorded by symmetrical elevation profilometers to a "true pavement profile." The advantages of using a "true pavement profile" as a basis for performance evaluation of pavements are discussed and examples are presented.

The feasibility of extending this procedure to obtain a continuous load-deflection profile and the modification of presently available equipment to accomplish this objective are discussed.

•AN ELEVATION PROFILOMETER measures and records continuously the deviation of the road profile from a selected baseline or local reference elevation. This local reference elevation is established from the weighted average of pavement elevations at predetermined distances on each side of the recording wheel.

The following procedure is designed to convert this recorded profile to an elevation profile, which has as a reference only a few true elevations (control elevations) widely spaced along the profile at convenient intervals.

Because a satisfactory description of most highway profiles may be obtained by taking elevation measurements at intervals of 1 ft along the pavement, this spacing has been selected for the computed elevations.

The geometry of the University of Michigan profilometer has been used in the development of the procedure, but nearly identical procedures may be established for most profilometers with symmetrical geometry.

The University of Michigan Profilometer

The geometry of the University of Michigan profilometer is shown schematically in Figures 1 and 2. This profilometer is furnished with two independent recording units that permit simultaneous observation of both wheelpaths. Each unit consists of two sets of reference wheels (bogies) 30 ft apart and a recording wheel midway between the bogies. The resulting recording (Fig. 3) gives the difference between the elevation of the recording wheel and the weighted average of the elevations of all eight reference wheels as the profilometer travels along the pavement.

It has been demonstrated in previous surveys that the modified elevation profile, recorded by this profilometer, can be computed from true elevations taken at appropriate intervals on the pavement. From this it follows that true elevations on the pavement can be computed from the recorded profile, provided the appropriate boundary values (control elevations) are available. There is complete freedom in selection of these control elevations along the road profile in question because the recorded profile is continuous.

Paper sponsored by Committee on Pavement Condition Evaluation.

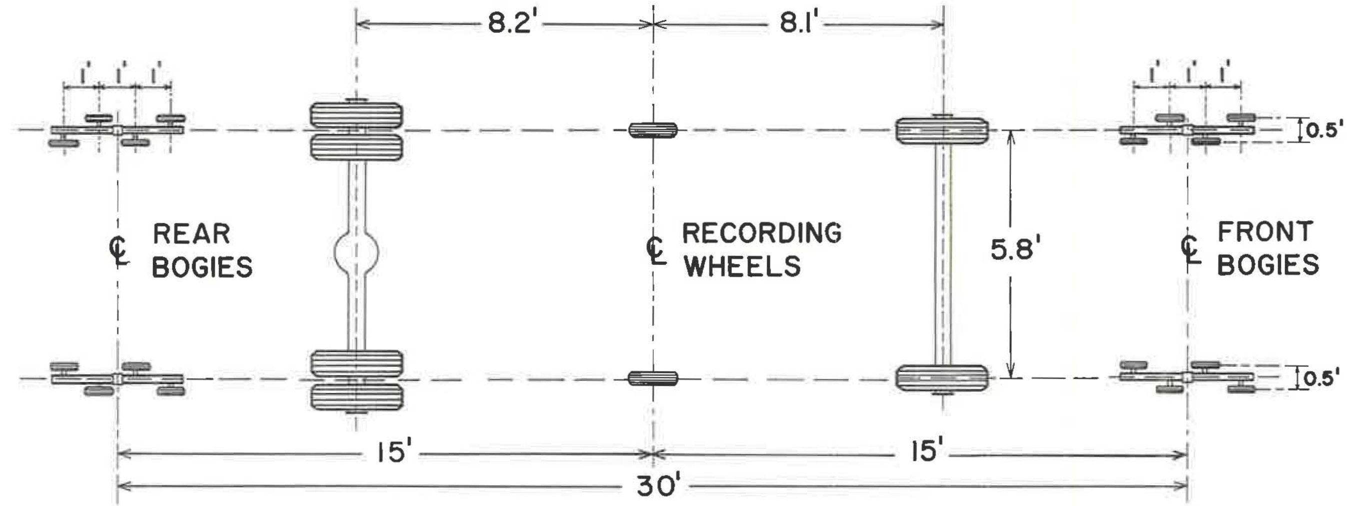

Figure 1. Geometry of University of Michigan profilometer—plan.

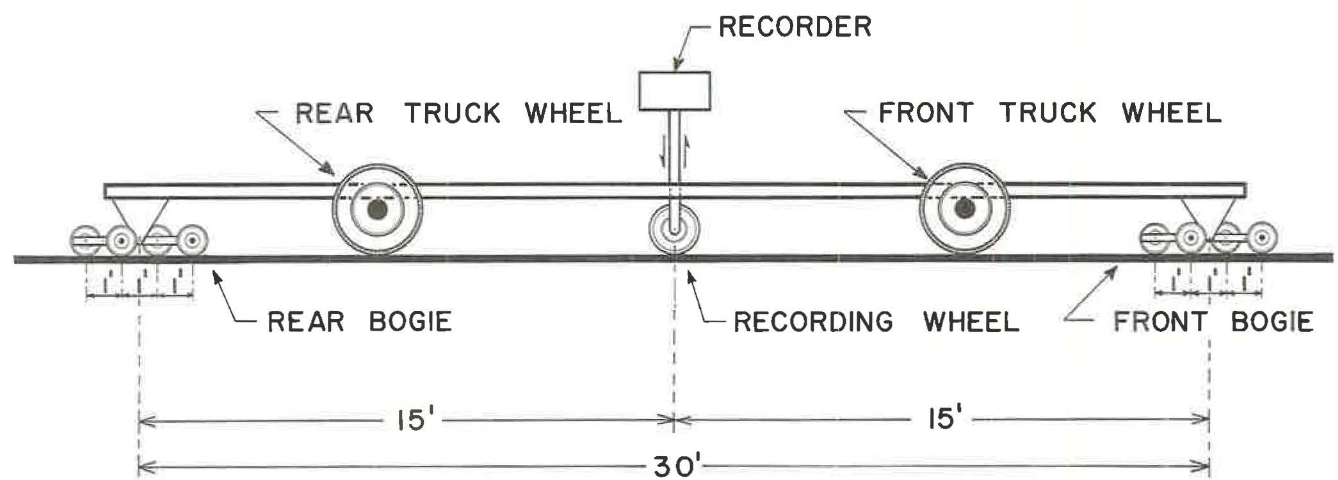

Figure 2. Geometry of University of Michigan profilometer—elevation.

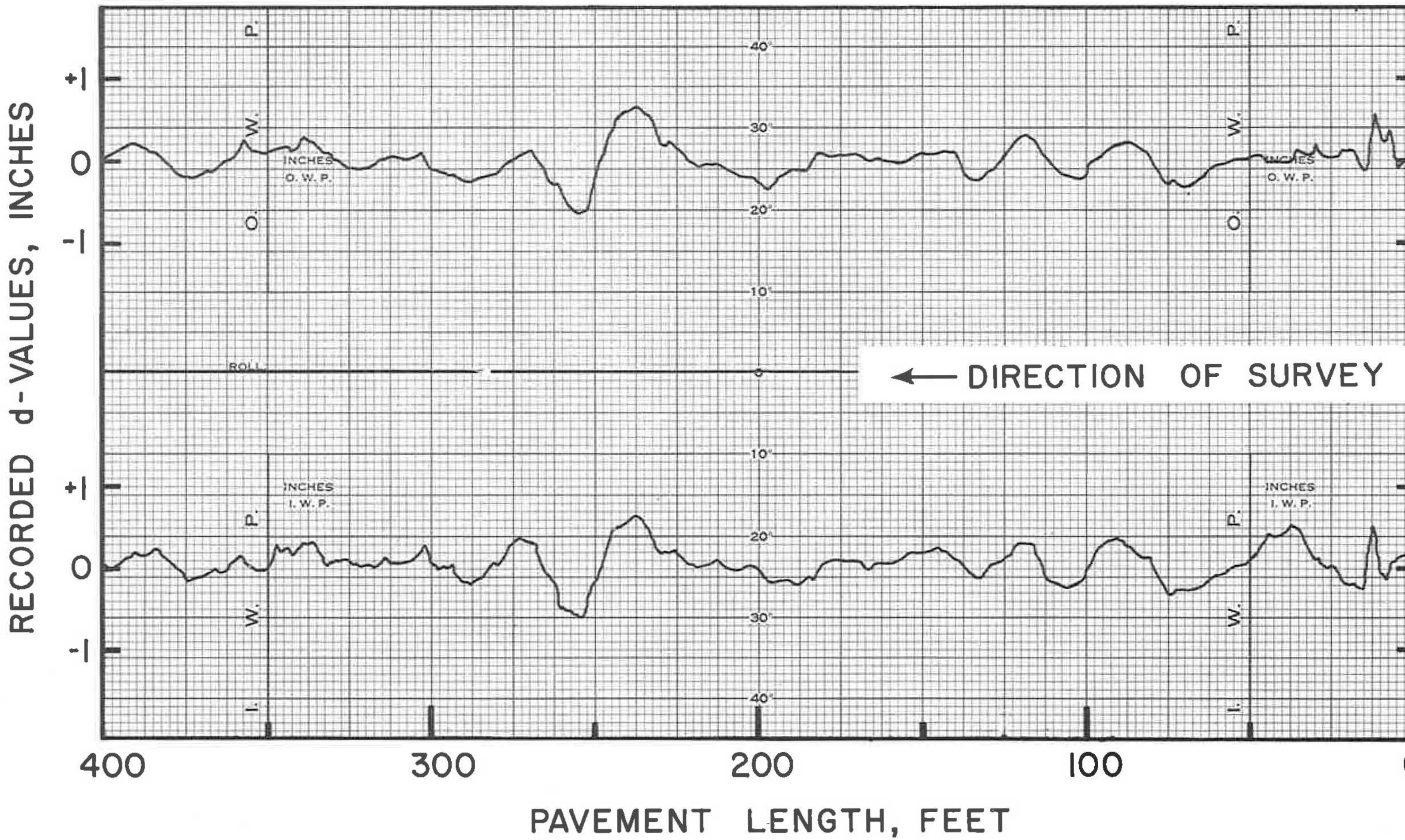

Figure 3. Typical profilometer recording.

Assumptions and Data

All wheels on the profilometer are assumed to respond similarly to irregularities in the road profile and to accurately reflect the actual pavement elevation under their axles.

Figure 1 shows that the recording wheel and the reference wheels of the University of Michigan profilometer do not lie on a straight line. For the purpose of the present development, it must be assumed that the weighted average elevation of the eight reference wheels in each recording unit would be the same if they were brought in line with the recording wheel or, if not, that any discrepancy would remain constant for the profile between two consecutive control elevations.

In using digital computers to carry out the numerical work in the profile conversion, it is not possible to use the continuous profilometer record; however, values corresponding to every foot of profile length are furnished as data.

Because the spacing of computed elevations is limited to 1 ft, and the distances between the wheels of the University of Michigan profilometer are integer multiples of no unit larger than ½ ft, it is necessary to interpolate between two elevation points when the reference wheels are located between them.

A minimum of three control elevations along the profile are also required for the profile conversion. These can be chosen at convenience along the pavement profile, except for the first and last points that must be more than 100 ft from the beginning and end of the profile, respectively.

Length measurements along the pavement profile are not discussed in detail here. However, it is assumed that the horizontal scale on the profilometer recordings is based on the length of a smooth line following the general surface of the road profile.

Outline of Analytical Procedure

To begin the analytical procedure, the relative position of any three consecutive elevation points on the profile, expressed in terms of the difference between the elevation of the center point and the average of the others (D-values), must be estimated. This is accomplished through a series of iterative computational steps that require as data only the relative values from the profilometer record (d-values).

Having obtained D-values for the entire profile, it is now possible to deduce a new record of d-values (d′-values), which would have been obtained had the profilometer been equipped with single wheels instead of the multiwheel bogies.

Next, the lengths between the various control points are adjusted so that these are spaced at distances that are integer multiples of 15 ft (half the profilometer length). In the beginning of the computation, the numerical values of the control elevations are not adjusted accordingly, but as the relative elevations for the entire profile emerge with increasing accuracy, appropriate adjustments are carried out.

The zero baseline about which the computed d-values oscillate is now established. This is done by assuming it to coincide with the average of the d′-values in each interval between control elevations. Based on this assumption, a step-like projection is carried out using a simulated three-point profilometer. Starting with the rear reference wheel over a control point, the relative elevation under the two reference wheels and the central recording wheel is expressed in terms of the deduced d′-values. The profilometer is then moved toward the successive control points in jumps equal to half the length of the reference beam, but for each position the pavement elevation under the leading reference wheel is expressed as a function of the rear reference wheel, the central recording wheel and the pertinent d′-values. Because the profilometer wheels must pass through all control points, a sufficient number of conditions is established to permit evaluation of the zero baseline for the d′-values in each interval between control points. Further, by using the relationships already established between the d′- and d-values, the absolute values for the latter can also be deduced.

A similar projection procedure is now undertaken, but elevation values are established at 15-ft intervals for all but 50 ft at each end of the profile.

The intermediate elevations are obtained through a series of approximations. The absolute d- and D-values already computed are used in the first approximation. The compatibility of the elevation values obtained is checked and adjustments are made as required. These are made both directly on the elevation values by using relationships determined by the geometery of the profilometer and indirectly by reevaluation of the previously determined D-values.

When the entire profile has been determined within the limitations of the approximations made, adjustment is made in those control elevations that were shifted to conform with the spacing requirements.

The steps are repeated, both individually and in sequence, until the true elevation profile is obtained with as much accuracy as the data permit.

Use of High Speed Digital Computer

To accomplish the numerical work required in the conversion procedure outlined, a high speed digital computer must be used. A program has been written for this purpose, but, because of its length, it cannot be presented here. However, to demonstrate how this program works, two numerical examples are cited.

The computer program was tested for accuracy in establishing a true elevation profile from an exact d-profile and a few reference elevations by use of about 3,000 ft of measured highway elevations. The d-profile that, theoretically, would have been recorded by the present profilometer was computed from these elevation data and used as input for the computer. In addition, elevation values at 120, 720, 1,620, 2,220 and 2,760 ft were chosen to serve as control points. The entire elevation profile is shown in Figure 4 with the control points marked.

Most of the computed elevations obtained after approximately 1½ min of computing time differed from the original elevation values by less than 0.002 in. In the center of the 900-ft span this difference reached 0.005 in., and for 40 to 50 ft at each end of the profile, oscillatory errors as high as 0.030 in. were present.

The same stretch of road from which the elevation measurements used in the preceding example were obtained were also profiled by the University of Michigan. How nearly the path measured by the profilometer coincided with that from which the levels were taken is open to question.

However, the recorded profilometer data corresponding to the first 1,500 ft of the profile shown in Figure 4 were used as a basis for computation of the elevation profile by the present method. In addition to the values from the profilometer recording, five control elevations spaced at 300 ft along the profile were also selected.

A section of the computed elevation profile is shown in Figure 5. For comparison, the corresponding elevation profiles plotted from actual level measurements and the recorded d-profile are also shown. The section shown is that inclosed by a rectangular frame in Figure 4. The results are most encouraging, though there are some differences to be noted in the two elevation profiles. Those differences, which amount to a maximum of about 0.4 in., may be due to a number of factors. As mentioned previously, it is not likely that the path traveled by the profilometer exactly followed the points on which the rod was placed to obtain level measurements. The outside wheelpath along which the measurements were taken was partly on an extension of the old pavement and cracks that had formed between the extension and the original pavement reportedly interfered considerably with the profilometer wheels at the time of the survey. The observed drift in the recorded profile was quite uniform over extended stretches of pavement. In Figure 5 it is notable that this drift reversed at a point about midway of the sample shown. If the elevation measurements are reliable, this indicates that the profilometer wheels were influenced by the pavement condition to a different degree in the spans on each side of this point.

Advantages of an Elevation Profile

Because the profile recorded by an elevation profilometer is a function of both the character of the surface being measured and the geometry of the instrument itself, its usefulness is somewhat limited.

It can be shown, for example, that because of its geometry the University of Michigan profilometer responds very little to 15-ft waves in the road profile, whereas it records

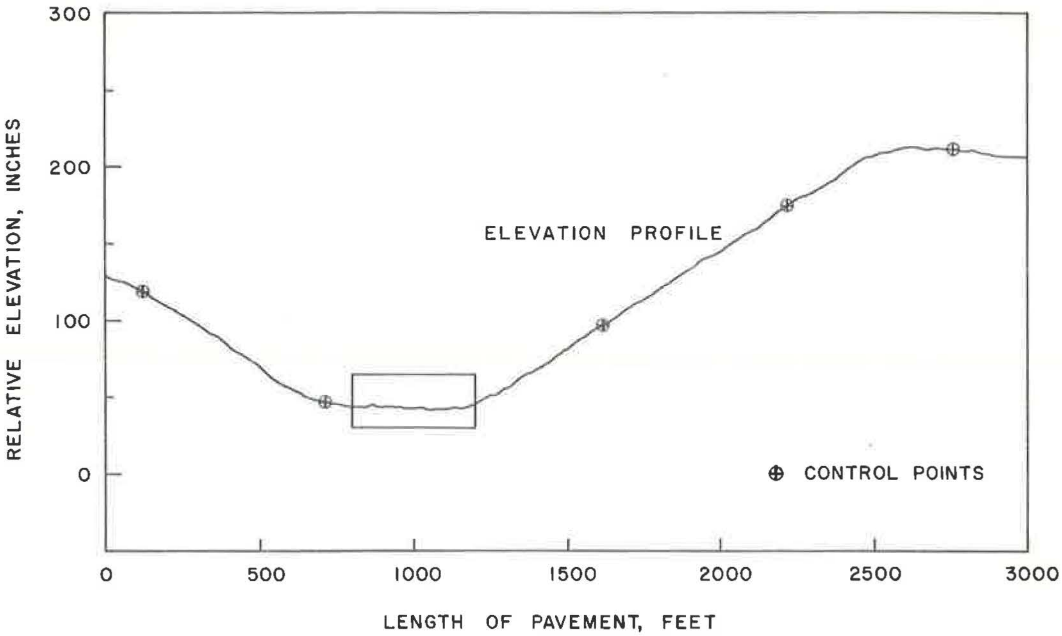

Figure 4. Elevation profile of test section.

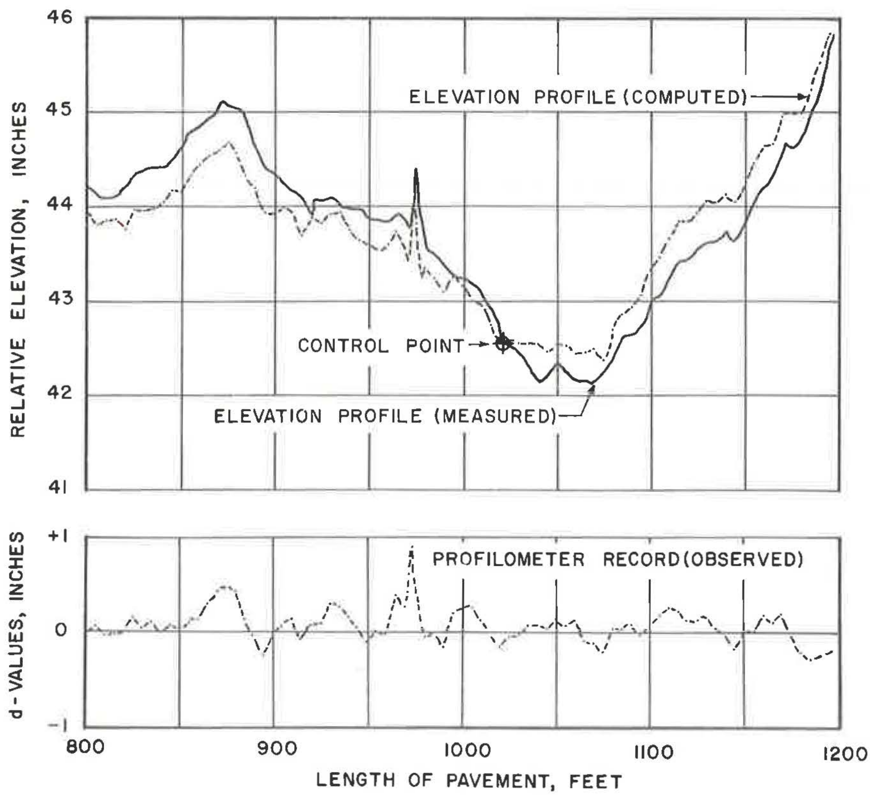

Figure 5. Comparison of computed and measured elevation profile.

almost double the amplitude of 30-ft waves. Undulations with regular periodicity obviously do not occur on actual pavements, but components of periodic waves do exist in any irregular surface and, hence, records obtained may be misinterpreted.

To illustrate the influence of the method used to observe a pavement, a section of an actual elevation profile obtained on US 20 in Indiana is shown in the upper part of Figure 6. An average grade line profile was superimposed on the measured elevation profile; subsequently, the difference in the elevation of these two profiles was computed

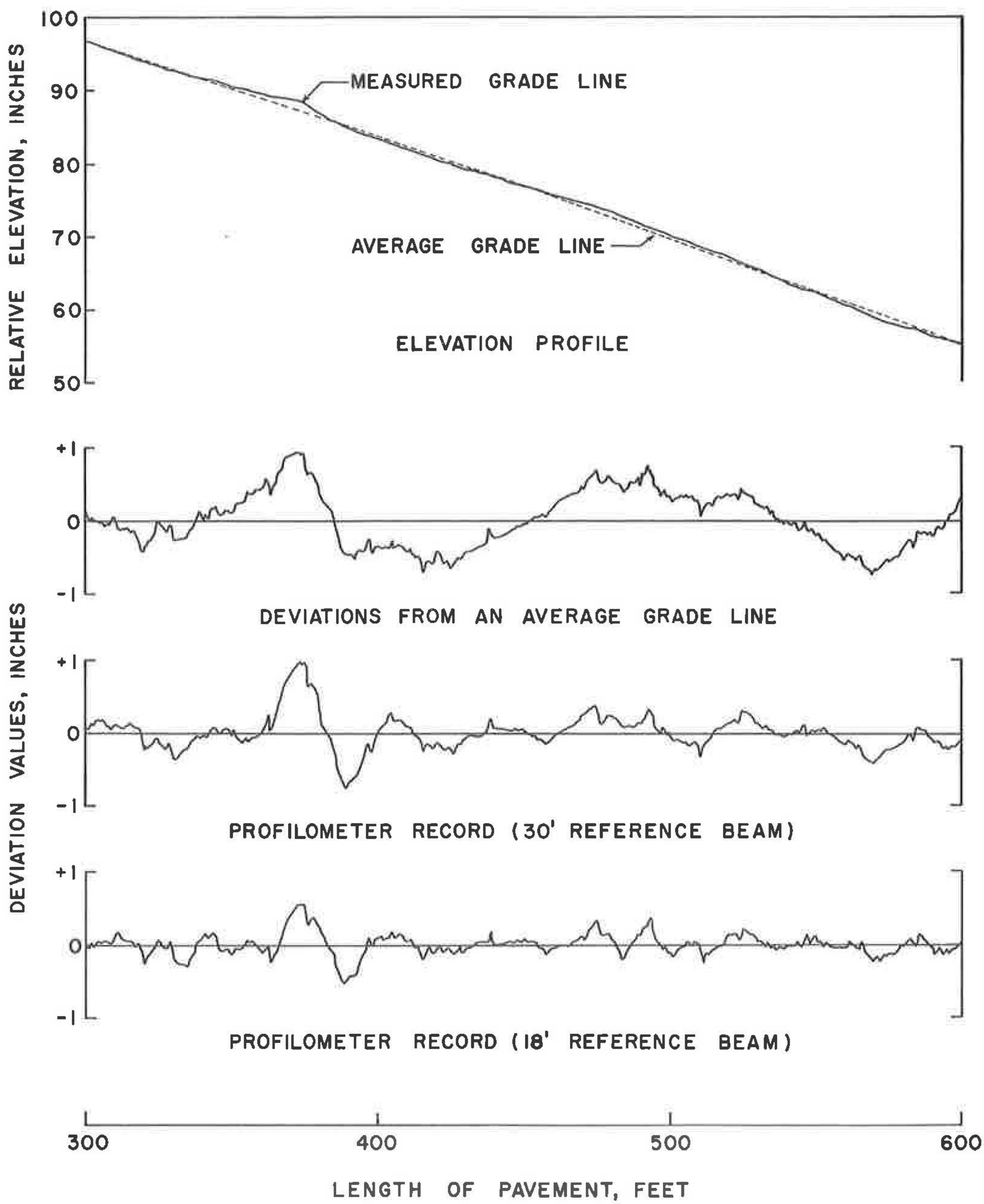

Figure 6. Examples of various deviation profiles.

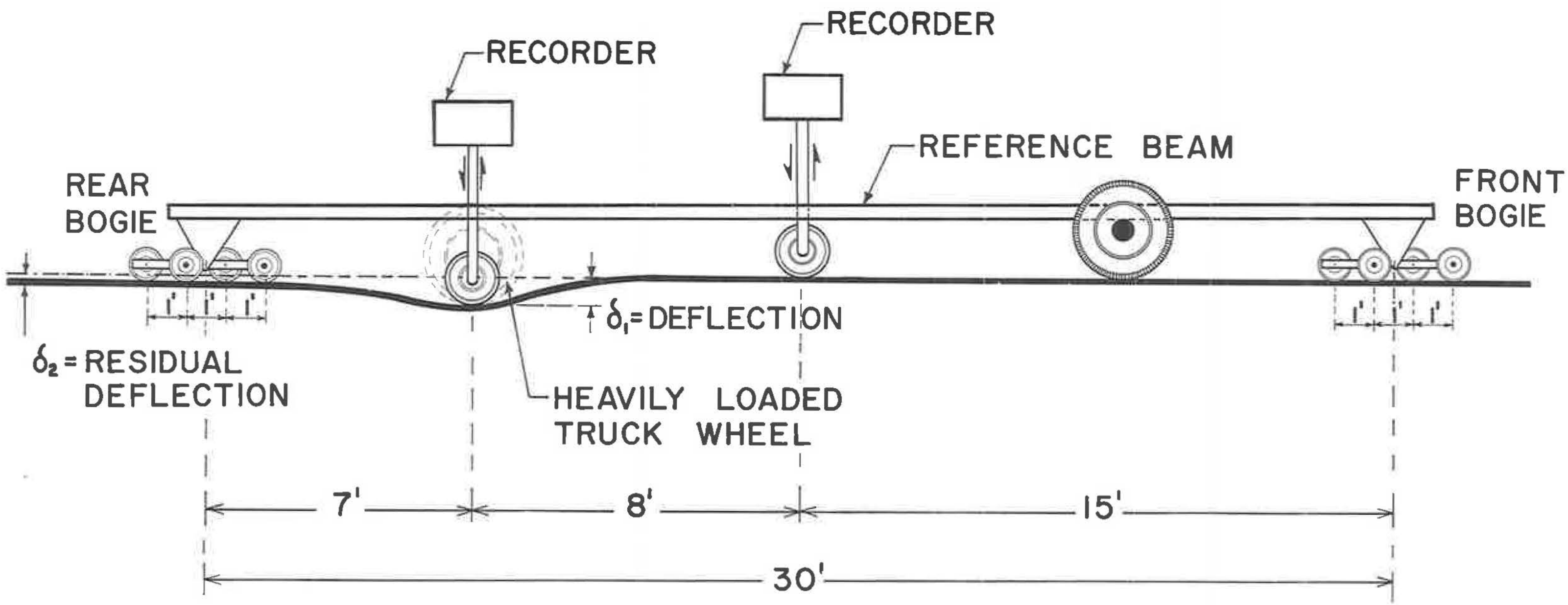

Figure 7. Schematic drawing of present profilometer modified to measure pavement deflection.

and plotted on the same graph. In the lower part of Figure 6 two recordings of d-values are shown. The first of these is obtained by a profilometer of the same geometry as that of the University of Michigan profilometer, whereas the second recording results if the reference beam of the present equipment is shortened to 18 ft. The three deviation profiles thus obtained for the same section of pavement clearly give different pictures of the character of the pavement roughness. Not only is there a difference in the indicated character of the pavement roughness at any one time, depending on which method of observation is used, but also the same influence is present when the changes in the character of the pavement roughness from one time to another are evaluated by the preceding means.

A true elevation profile computed from an elevation profilometer record greatly facilitates the study of specific pavement defects, because it is free of the distortion inherent in the profilometer record. In road-vehicle dynamic studies, the true profile is invaluable and can, for example, be subjected to power spectra analysis.

A commonly used measure of the riding quality of a pavement is the "Roughness Index." This index is based on the sum of the vertical changes in the profilometer record and is usually expressed in inches per mile of profile. A more rational measure of the condition of a pavement may be obtained if the plan grade to which the pavement was presumably built is used as a reference and the deviations of the true elevation profile are computed from it. The sum of the changes in this value could be expressed numerically in terms of inches per mile. If "as built" plans or grades are not available, average grades can be computed from the true elevation profile and then used as a reference as in Figure 6.

Deflection Measurements

The prospect of being able to compute a true elevation profile from profilometer data has also opened the door to the possibility of obtaining continuous deflection measurements along the road surface.

This may best be seen by considering Figure 7 in which the present profilometer equipment is shown schematically, but with a second recording unit added to it. If this second recording wheel is located so that it is always in contact with the pavement between the heavily loaded rear dual tires of the profilometer truck, the deviation values obtained by this unit with respect to the profilometer beam are a direct reflection of the elevation of the pavement under the heavy load. Because the elevation of the reference beam is fully determined from the elevation values computed from the regular recording, the second recording may be used directly to compute the elevation under the load. By then subtracting the two elevation values obtained for each point on the pavement profile, a continuous deflection profile can be produced.

As indicated in Figure 7, it is likely that the loaded truck wheel influences the other wheels of the profilometer. This is particularly true with respect to the rear bogie wheels, because any residual deflection due to the passage of the heavy truck wheel directly affects their elevation. A design in which the second recording unit is placed on a cantilevered extention of the profilometer beam would eliminate this difficulty. In this case the heavy wheel load can be located far enough behind the rear bogie wheels so as not to influence their elevation.

Because the deflection caused by a heavily loaded truck wheel is only 0.005 to 0.100 in. great precision is necessary in all the measurements performed. There do not seem to be any theoretical difficulties in acquiring this required precision, but there are undoubtedly many technical ones. It is, however, worthwhile to keep in mind that errors in individual deflection values may not be of much concern because the primary interest is to obtain a statistical summary of the deflection characteristic of a pavement over a limited length rather than precise values for individual points of the pavement.

CONCLUSIONS

The procedure outlined here may be successfully applied in converting profilometer records to true elevation profiles. These profiles are of great value not only for the purpose of assessing pavement riding quality but also for studying specific pavement defects by, for example, comparing a succession of true elevation profiles.

Further, it may be possible to build a profilometer that can be used to obtain data both for computing the true elevations along the pavement and the deflections under a loaded axle. Such an instrument, however, must be equipped with sensing devices and recorders far more accurate than those currently used. Preliminary studies indicate that instruments and parts meeting the required criteria are presently available.

ACKNOWLEDGMENTS

The subject matter presented here was taken from the author's research in pavement performance, portions of which will be included in a thesis entitled "Quantitative Evaluation of the Performance of Highway Pavements" to be submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Support for this research program has been mainly provided through the Michigan Pavement Performance Study under supervision of Professor W.S. Housel of the University of Michigan. Other agencies collaborating in this study include the Michigan State Highway Department in cooperation with the U.S. Bureau of Public Roads.

Grateful acknowledgment is extended to Professor Bayard E. Quinn of Purdue University for providing the elevation data on US 20 in Indiana.

Evaluation of Pavement Performance Related to Design, Construction, Maintenance and Operation

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This paper presents a summary of the findings of the Michigan Pavement Performance Study, a five-year program (1958-1963) to evaluate pavement performance from field surveys of existing pavements. In the five-year period, equipment and procedures for recording and analyzing pavement profiles have been developed and profiles of 10,000 miles of pavement have been accumulated. Although there have been several published reports of this work as it progressed, the final results have now been compiled and analyzed.

The quantitative evaluation of pavement condition and performance and the physical inventory of existing roads provide factual information of direct value in design, construction, and maintenance of both rigid and flexible pavement and in the operation of the state trunkline system as a transportation facility. The findings of the five-year study are reviewed, the adequacy of Michigan design standards is evaluated, and the effect on performance of certain construction practices is pointed out. The use of pavement profile data in more effective and timely maintenance and their value in the operation of the state highway system are discussed.

•STARTING in 1952, the University of Michigan and the Michigan State Highway Department have undertaken several projects in their cooperative highway research program in which special attention has been given to field surveys of existing pavements as a basis for evaluating pavement performance related to the design, construction, maintenance, and operation of highway pavements. At that time, in cooperation with the Wire Reinforcement Institute, a five-year survey was initiated to study the effect of steel reinforcement on the performance of concrete pavements.

Condition surveys of existing pavements as a check on design and a basis for more effective utilization of natural conditions and materials in highway construction are not new. As pointed out in several of the current references, this approach had been used for many years and was the fundamental basis for Michigan design of the roadway structure. As part of the study, definite criteria for measuring pavement performance in a quantitative manner had been set up and put into practice under field conditions (1, 2). The primary function of a pavement is to provide a smooth riding surface, supplying safety, comfort, and economy to the highway user. Recognizing this, riding quality has been defined in terms of a roughness index, RI, expressing the cumulative or total inches of vertical displacement per mile measured from the recorded pavement profile.

It was also recognized that the structural properties of the pavement would control its ability to endure under the combined stresses of continuous load repetition and the rigors of its environment. It seemed logical that failure to survive or inadequacy as a structure would be reflected in cracking or loss of structural continuity even before

Paper sponsored by Committee on Pavement Condition Evaluation.

riding quality was affected. Timely maintenance or corrective steps would depend on early identification of weakness, so a continuity ratio was adopted as an independent quantitative measure of structural adequacy. The continuity ratio was defined as the ratio of the uncracked slab length of a pavement divided by 15. The control length of 15 ft was selected as a measure of the normal subdivision of a rigid concrete slab due to shrinkage, warping, and curling under temperature, moisture, and other environmental influences. It was considered that such environmental effects did not reflect structural inadequacy; thus, slab lengths of 15 ft or more would not be considered evidence of structural weakness.

The adoption of these criteria and their application to condition surveys of existing pavements in the early 1950's was not a generally recognized approach, and represented simply an attempt to quantify in some integrated form the many factors which affect pavement performance. Others were concentrating their efforts on the road test approach and the AASHO Road Test was then in the planning stage. As late as 1958, field surveys were not being considered as an alternate to the satellite tests in the recommended procedures then being circulated to follow up the AASHO Road Test.

The second phase of the Michigan investigation began in 1957 as a cooperative program with three agencies of the trucking industry, The Michigan Trucking Association, The American Trucking Associations, Inc., and the Automobile Manufacturers Association. After the first two years, the Michigan Pavement Performance Study, as it was known, was taken over more directly by the Michigan State Highway Department, as part of the Michigan Highway Planning Survey Work Program HPS-1, in cooperation with the U.S. Bureau of Public Roads. Acknowledgment of the contributions to the study of a number of organizations and many individuals was made in the "Five-Year Summary of the Michigan Pavement Performance Study," prepared after termination of the study in December 1962. The current paper is a resume of that "Summary," intended to provide reference to the complete series of reports which accompanied it and to abstract from it the more important findings.

Evaluation of pavement performance on a large scale by the procedures used in this investigation was undertaken in the belief that carefully controlled observations of existing pavement under actual service conditions and environment would provide the answers to some of the most perplexing problems facing the highway engineer. The first year, from September 1957 through the first half of 1958, was devoted largely to selecting procedures and designing, planning, and assembling equipment. In spite of a disappointingly long shake-down period for the truck-mounted profilometer, considerable mileage of pavement profile was recorded in the first year and a half.

In the five years that the Michigan Pavement Performance Study was in progress, profiles of almost 10,000 lane miles of pavement were recorded. The annual totals are given in Table 1. On some routes only one lane has been surveyed, normally one of the traffic lanes. On a large part of the mileage, particularly on new construction and certain roads of special interest, all lanes, including both traffic and passing lanes, have been surveyed.

This large mileage of recorded pavement profile and supplemental data represent a volume of basic information on pavement condition and performance, the value of which has been only partially utilized to date. This review illustrates the use of this information in design, construction, maintenance, and operation of the Michigan trunkline system. However, its value as a pavement inventory and a foundation on which to build future applications of practical value can be realized only by its continued use and by keeping it up-to-date and growing as the highway system grows.

Having established criteria and general procedures for the pavement performance surveys, the truck-mounted profilometer with its electronic recording instrumentation was developed as the major piece of equipment. It was modeled after that designed by F.N. Hveem and used by the California Department of Highways. It was selected as the most practical under field conditions and state highway department operation to collect and record a large volume of pavement profile data. Many types of road roughometers have been described and used with varying success, but the choice had to be made from those which were readily available. There was little time to devote to devising and developing instrumentation; the California machine was operating efficiently and, with some modifications, met the needs of the Michigan study.

Modifying the California equipment for recording a continuous pavement profile in only one wheel track, a double recording system was adopted which provided profiles in both the outer and inner wheel paths in one operation. Electronic integrating instrumentation was added to record the cumulative roughness in inches of vertical displacement for each quarter mile. More details on the profilometer and its operation are given in previously published reports included as part of the five-year summary.

TABLE 1

TOTAL MILEAGE OF RECORDED PAVEMENT PROFILE

Year	Mileage
1958	1,969.2
1959	2,128.2
1960	1,769.4
1961	2,366.8
1962	1,535.6
Total	9,769.2

UTILIZATION OF PAVEMENT PROFILE DATA

The title of this paper indicates that pavement performance data find application in design, construction, maintenance, and operation of highways. It is not always recognized that a highway department actually has four major functions which may be so delineated in describing different phases of its operations. However, in planning this review of the pavement performance study, it appeared not only appropriate but also necessary to so classify highway activities in order to accurately illustrate the usefulness of pavement profile data.

Design Correlation

The primary objective of the Michigan Pavement Performance Study was to provide more accurate and discerning techniques for checking pavement design and detecting weakness in service performance. It seemed entirely logical that changes in the pavement surface or profile would reflect the integrated result of the various stresses and strains to which a pavement is subjected, originating from variations in the supporting subgrade below or from repeated load application and weather cycles above. Although the uncontrolled variables of environment seem much more difficult to gage than the more precise relationships of applied load and reaction in the pavement structure, they are nevertheless the influences under which pavements must endure. Every one of these variables, controlled or uncontrolled, has its effect on the pavement surface; whether or not they can be identified is a test of the observer and the methods of analysis brought to bear on the problem.

At first it was thought that an initial reference profile would have to be recorded and then, after a sufficient period of time had elapsed to produce a measurable change, a subsequent profile would measure the change. This meant that a period of years, perhaps many, would be required before definitive changes would become apparent. It came then as an unexpected bonus when, after a considerable volume of profile data had been accumulated, it turned out that roads, which had been in service for varying periods of time under varying conditions of service and environment, fell into definite patterns of behavior that could be defined in terms of pavement roughness, structural continuity, and related characteristics of the pavement. This discovery opened the door to a great storehouse of valuable data when it became apparent that the entire highway system was the final testing ground and that the many years these roads had already been subjected to traffic was the ultimate road test and was merely awaiting analysis.

From the standpoint of pavement design, the reports referred to herein contain many examples in which the responsible factors in pavement performance have been clearly identified in terms which demonstrate them to be subject to design control. The overriding importance of soil conditions and drainage stands out in many of these examples and demonstrates the soundness of Michigan design, which follows the unspectacular but time-tried principle that it is the subgrade that "does, in fact, carry the road and the carriage also."

TABLE 2

Route No.	Service Period (yr)	Roughness Index (in. per mi)		Drainage	Riding Quality
		OWP	IWP		
US-112	32	72	75	Fair	Very Good
	32	291	395	Poor	Prohibitive[1]
M-25	28	91	66	Excellent	Good to very good
	36	218	175	Poor	Very poor
M-41	22	383	365	Poor	Prohibitive[1]
	22	73	75	Good	Very good
M-36	19	84	77	Excellent	Good
	19	363	282	Poor	Prohibitive[1]

[1] Outside tentative rating scale.

A few illustrations drawn from the supplementary reports to the five-year summary may be cited for illustration. Several of these reports are included in the list of references for this paper and in the "Five-Year Summary of the Michigan Pavement Performance Study" as Publications and Papers, identified as Report Series P (P-1 through P-6). Table 2 lists the correlation between riding quality and drainage previously reported (3). It should be noted that drainage as listed includes internal drainage as controlled by soil texture and ground water level.

Other design correlations reported (3) include the poor performance of short concrete slabs without load transfer at the joints (Fig. 1, top), as compared to the performance of another road also having comparatively short slabs, but with load transfer provided (Fig. 1, bottom). Although there were other factors involved to some degree, the contrast between these two roads was so sharp that the comparison is still valid, with the first pavement becoming extremely rough in its period of service and the second maintaining very good riding quality over a considerably longer period of time.

The most interesting feature of the rough pavement in the preceding example is the characteristic saw-toothed pattern produced by tilting and faulting of the short slabs. This illustrates the unique value of an actual pavement profile that goes beyond the RI derived from it. Such a profile is a realistic picture of the pavement itself and the physical condition produced by some specific factors among a variety of influences that may have been present. Such a profile is as individualistic as a signature, reflecting characteristics that can be fully appreciated only by examining the profile itself and the physical conditions associated with it in whatever detail is necessary to read the pavement's past history.

This leads to perhaps the most important consideration in evaluating pavement performance from condition surveys. The RI or some other quantity derived from the pavement survey may adequately reflect the present riding quality or serviceability of the pavement. This in itself is an important consideration and may be useful in several respects. However, from the standpoint of pavement design, one must know not only the extent to which a pavement has deteriorated or lost riding quality but why it has reached that particular level of serviceability. This is the crux of the situation and the point at which the actual pavement profile shows its real value, as it may provide an insight into events in the past history of the pavement which have left no other clues (4).

An interesting comparison is provided in Figure 2 which demonstrates not only the necessity for detailed study of the profile, but also its intimate relation to the pavement itself, which must also be taken into consideration. In this section, the profile in the outer wheel path again shows a saw-toothed pattern, almost identical to that caused by the tilting of short slabs; one might be tempted to conclude that this pattern of displacement would certainly be due to the tilting of short slabs, except that the inner wheel path does not follow the pattern. Furthermore, the pavement is a 9-in. reinforced concrete pavement, 99 ft between contraction joints, and there are no cracks coinciding with peaks of displacement in the profile. Further investigation indicated that this was "built-in" roughness resulting from careless form setting, with displacement at the junction between 10-ft forms and sagging of the forms between points of support. This type of built-in roughness was most apparent in the outer wheel path, but also showed in the inner wheel path.

There are a number of other examples of the surprising consistency with which accurate pavement profiles and the quantitative criteria derived from them single out abnormalities in pavement behavior or unusual conditions which have affected pavement performance. For more complete study of all such information, reference should be made to the reports submitted as part of the five-year summary.

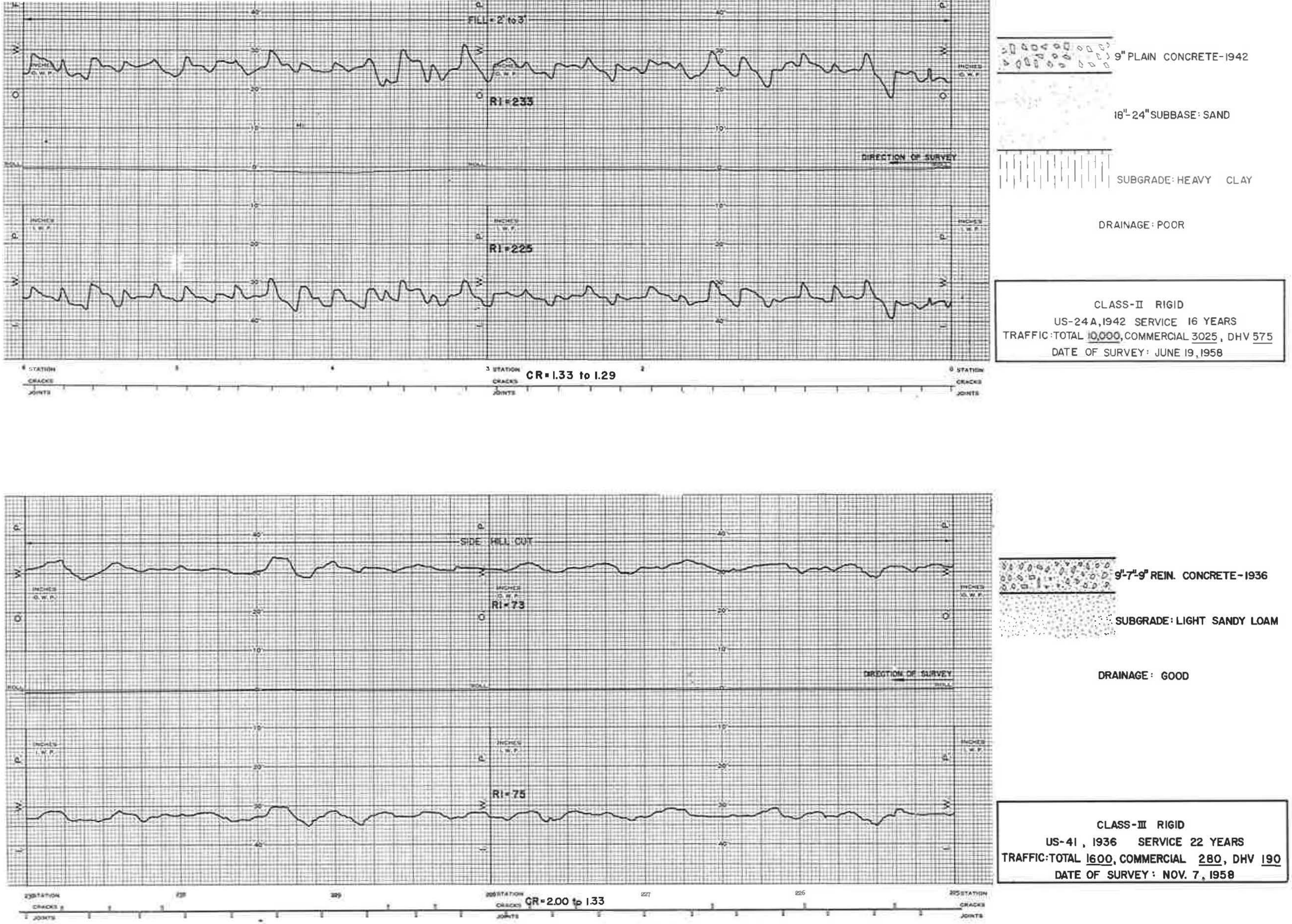
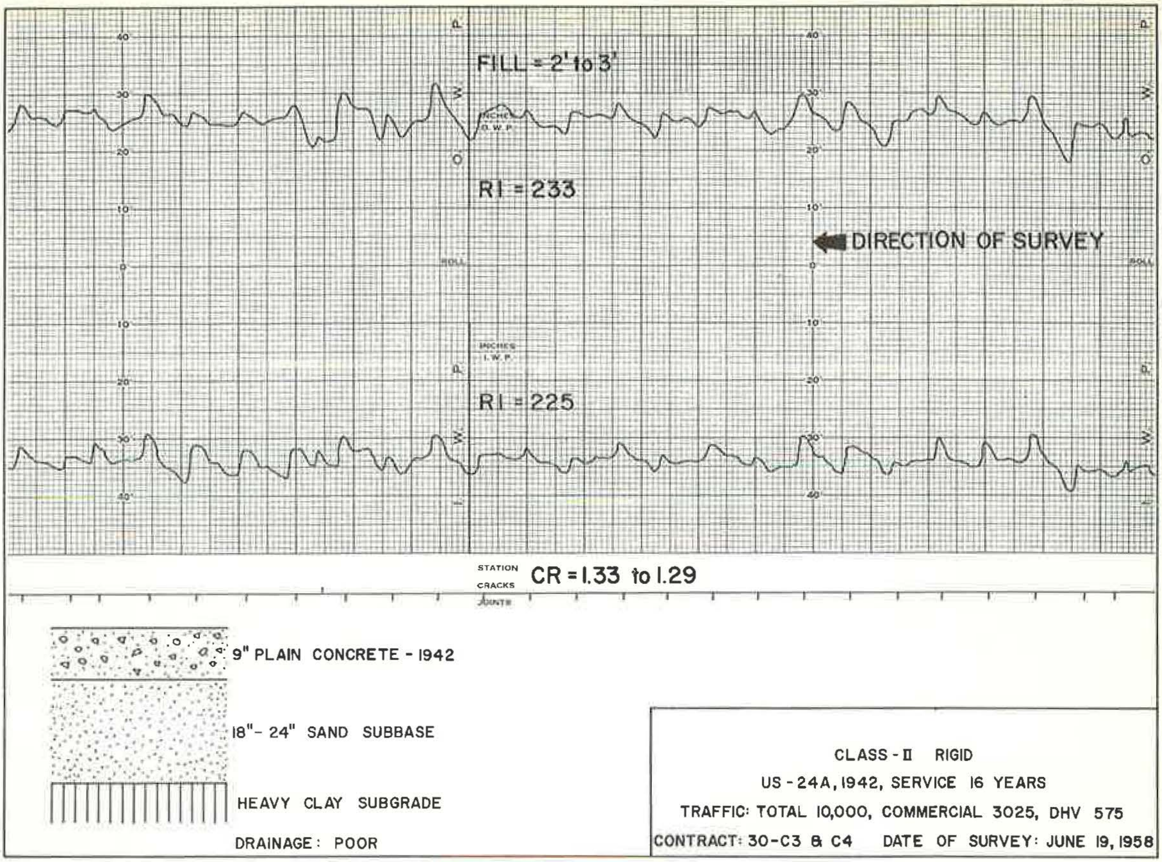

FILL - 2' to 3'
RI = 233
RI = 225
DIRECTION OF SURVEY
9" PLAIN CONCRETE-1942
18"-24" SUBBASE: SAND
SUBGRADE: HEAVY CLAY
DRAINAGE: POOR
CLASS-II RIGID
US-24 A, 1942 SERVICE 16 YEARS
TRAFFIC: TOTAL 10,000, COMMERCIAL 3025, DHV 575
DATE OF SURVEY: JUNE 19, 1958
CR = 1.33 to 1.29
SIDE HILL CUT
RI = 73
RI = 75
DIRECTION OF SURVEY
9"-7"-9" REIN. CONCRETE-1936
SUBGRADE: LIGHT SANDY LOAM
DRAINAGE: GOOD
CLASS-III RIGID
US-41, 1936 SERVICE 22 YEARS
TRAFFIC: TOTAL 1600, COMMERCIAL 280, DHV 190
DATE OF SURVEY: NOV. 7, 1958
CR = 2.00 to 1.33

Figure 1.


FILL = 2' to 3'
RI = 233
DIRECTION OF SURVEY
RI = 225
CR = 1.33 to 1.29
9" PLAIN CONCRETE - 1942
18"- 24" SAND SUBBASE
HEAVY CLAY SUBGRADE
DRAINAGE: POOR
CLASS - II RIGID
US - 24A, 1942, SERVICE 16 YEARS
TRAFFIC: TOTAL 10,000, COMMERCIAL 3025, DHV 575
CONTRACT: 30-C3 & C4 DATE OF SURVEY: JUNE 19, 1958

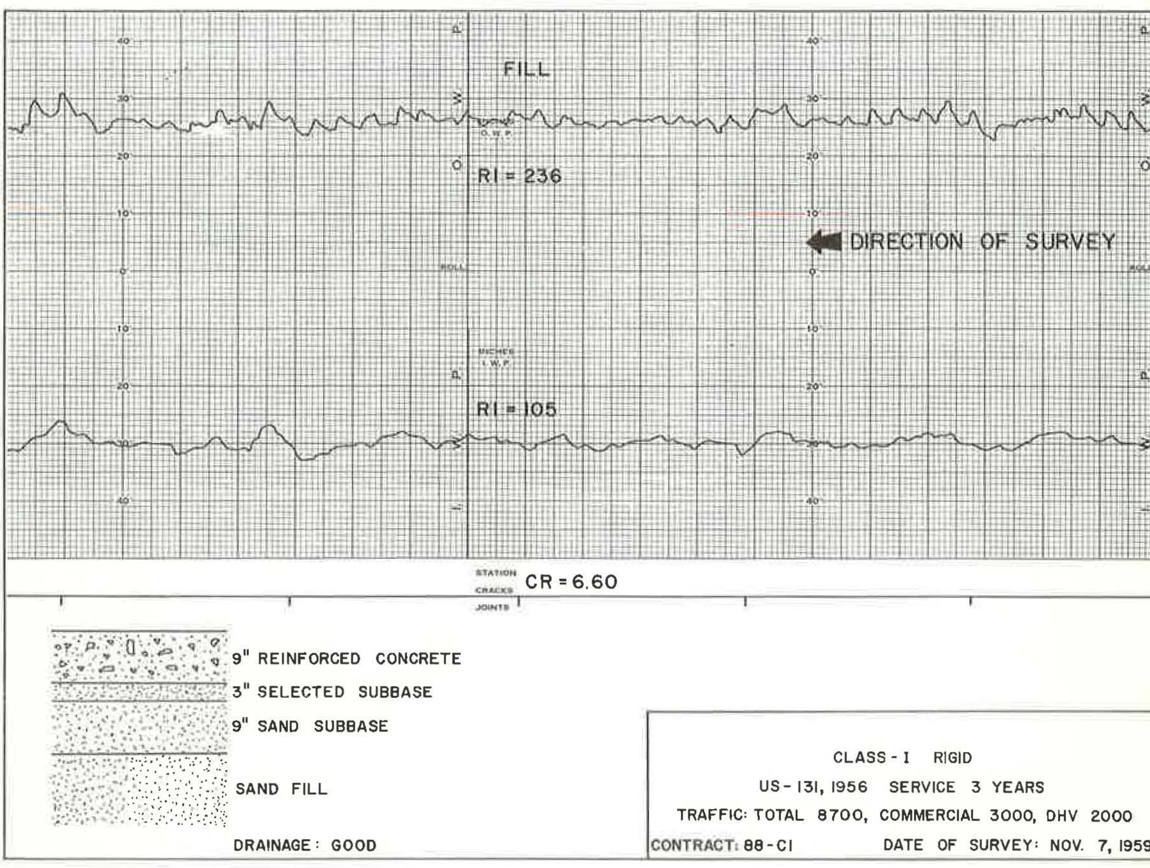

FILL
RI = 236
DIRECTION OF SURVEY
RI = 105
CR = 6.60
9" REINFORCED CONCRETE
3" SELECTED SUBBASE
9" SAND SUBBASE
SAND FILL
DRAINAGE: GOOD
CLASS - I RIGID
US - 131, 1956 SERVICE 3 YEARS
TRAFFIC: TOTAL 8700, COMMERCIAL 3000, DHV 2000
CONTRACT: 88-C1 DATE OF SURVEY: NOV. 7, 1959

Figure 2.

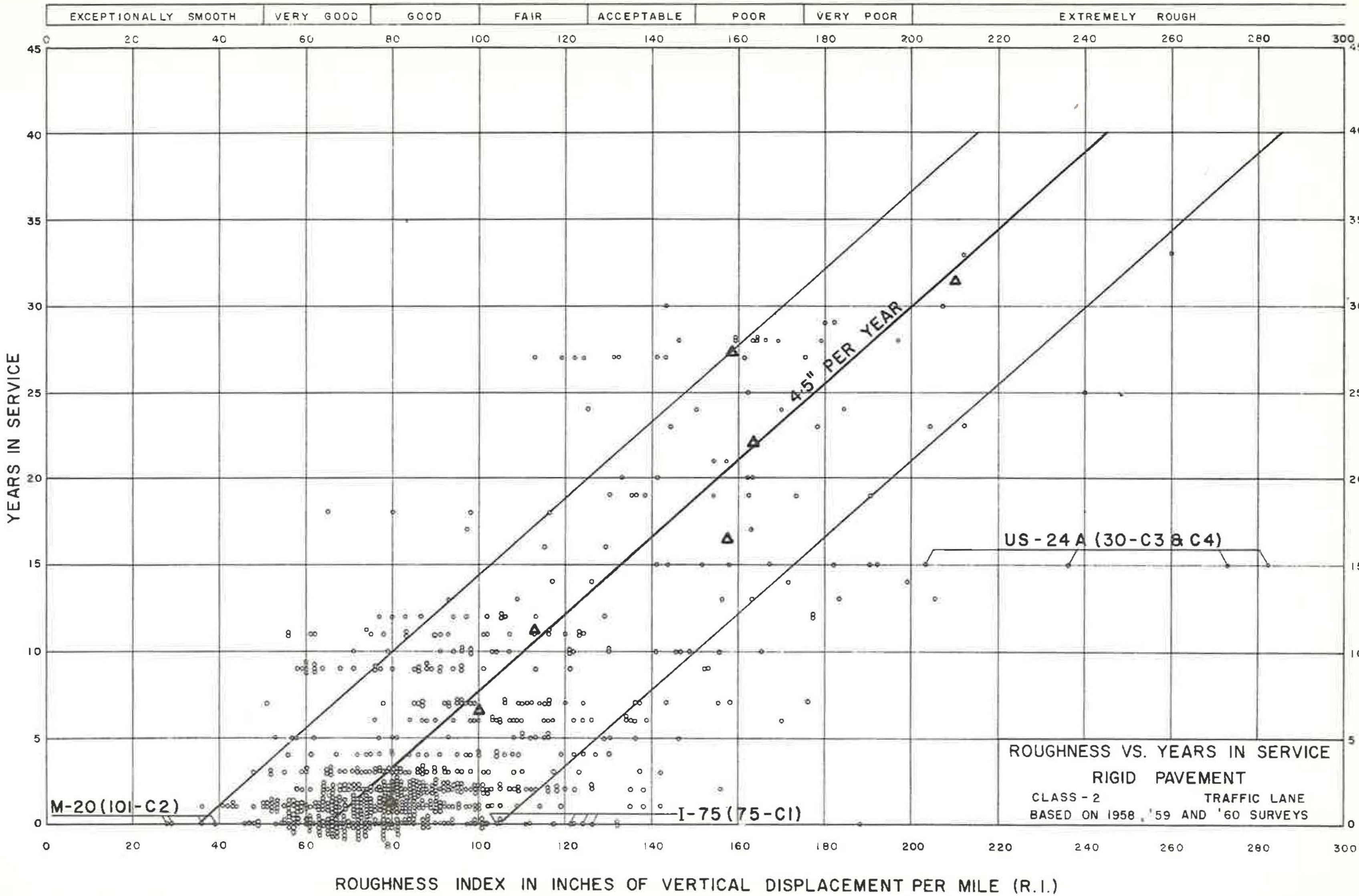

EXCEPTIONALLY SMOOTH
VERY GOOD
GOOD
FAIR
ACCEPTABLE
POOR
VERY POOR
EXTREMELY ROUGH
YEARS IN SERVICE
4.5" PER YEAR
US-24 A (30-C3 & C4)
M-20(101-C2)
I-75(75-C1)
ROUGHNESS VS. YEARS IN SERVICE
RIGID PAVEMENT
CLASS - 2
TRAFFIC LANE
BASED ON 1958, '59 AND '60 SURVEYS
ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

Figure 3.

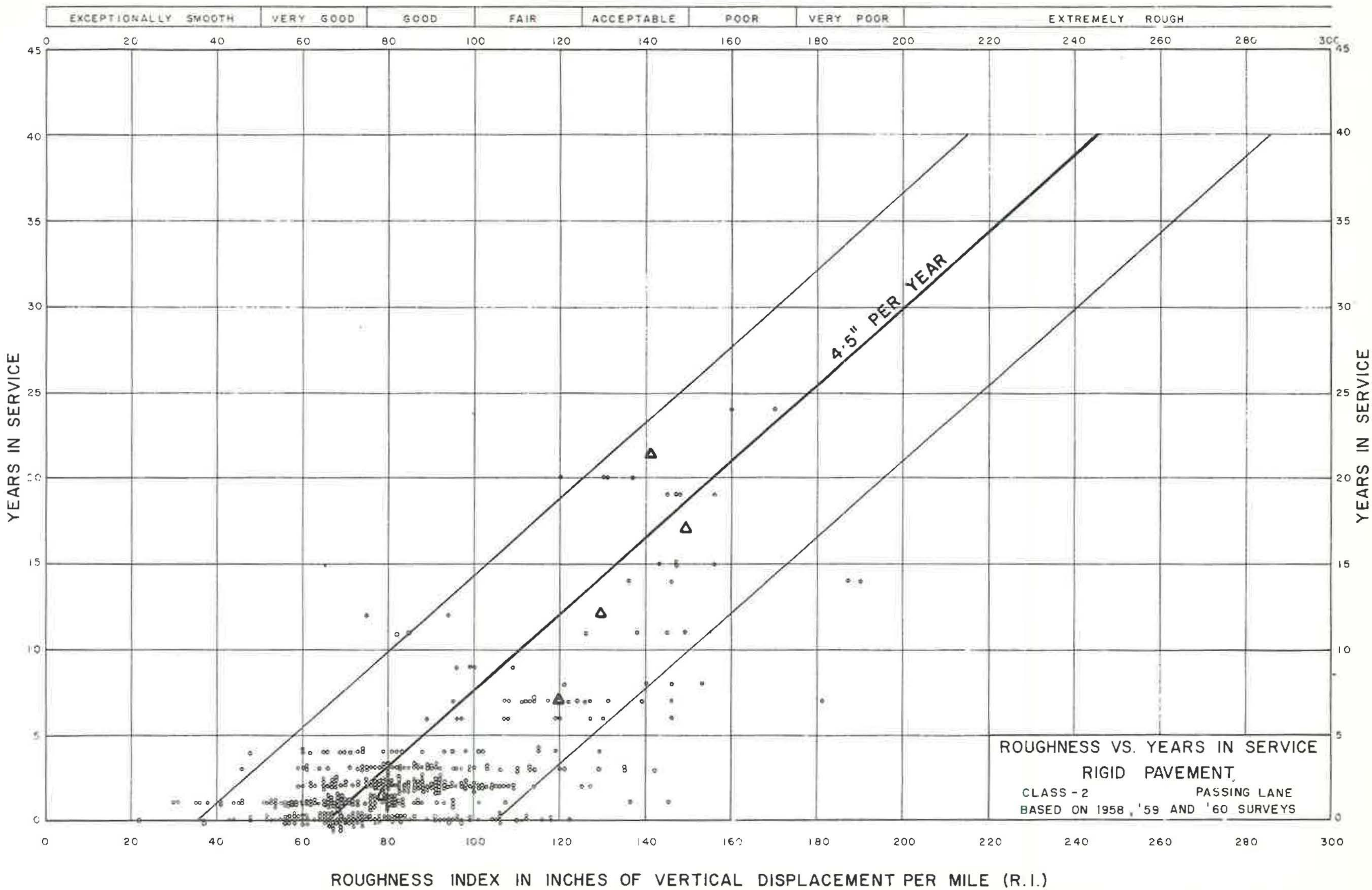

EXCEPTIONALLY SMOOTH
VERY GOOD
GOOD
FAIR
ACCEPTABLE
POOR
VERY POOR
EXTREMELY ROUGH
YEARS IN SERVICE
4.5" PER YEAR
ROUGHNESS VS. YEARS IN SERVICE
RIGID PAVEMENT,
CLASS - 2
PASSING LANE
BASED ON 1958, '59 AND '60 SURVEYS
ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

Figure 4.

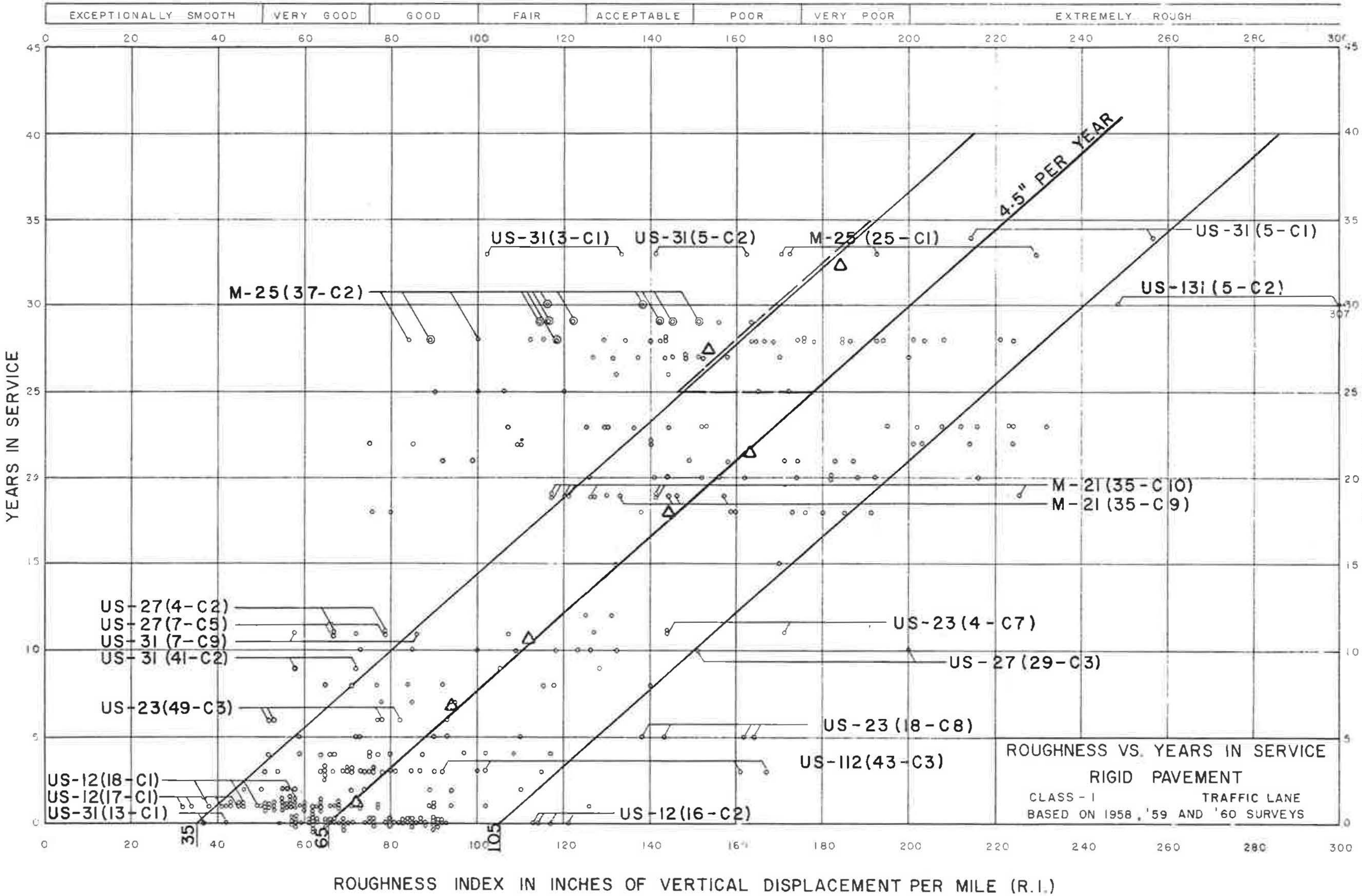
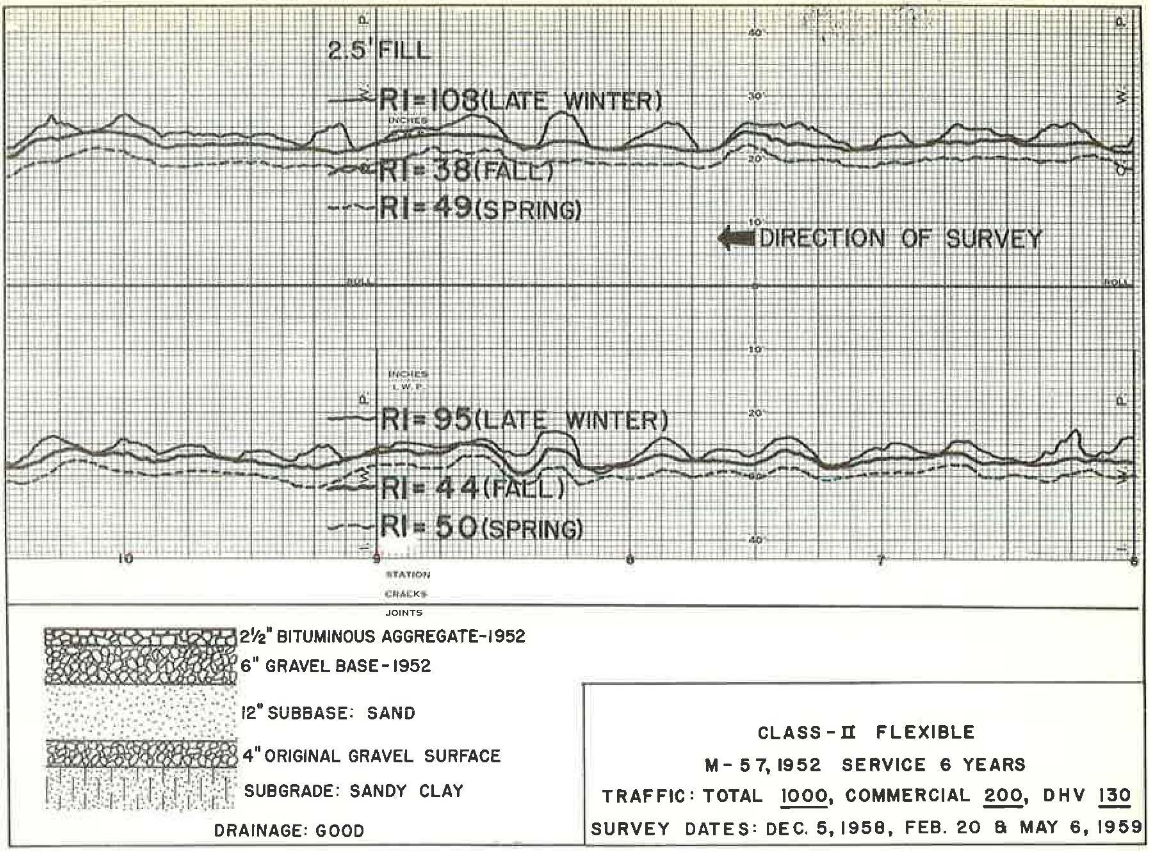

EXCEPTIONALLY SMOOTH
VERY GOOD
GOOD
FAIR
ACCEPTABLE
POOR
VERY POOR
EXTREMELY ROUGH
YEARS IN SERVICE
4.5" PER YEAR
US-31(3-C1)
US-31(5-C2)
M-25(25-C1)
US-31(5-C1)
M-25(37-C2)
US-131(5-C2)
M-21(35-C10)
M-21(35-C9)
US-27(4-C2)
US-27(7-C5)
US-31(7-C9)
US-31(41-C2)
US-23(4-C7)
US-27(29-C3)
US-23(49-C3)
US-23(18-C8)
US-112(43-C3)
US-12(18-C1)
US-12(17-C1)
US-31(13-C1)
US-12(16-C2)
35
65
105
307
ROUGHNESS VS. YEARS IN SERVICE
RIGID PAVEMENT
CLASS - I
TRAFFIC LANE
BASED ON 1958, '59 AND '60 SURVEYS
ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

Figure 5.


2.5' FILL
RI=108 (LATE WINTER)
RI=38 (FALL)
RI=49 (SPRING)
DIRECTION OF SURVEY
RI=95 (LATE WINTER)
RI=44 (FALL)
RI=50 (SPRING)
STATION
CRACKS
JOINTS
2½" BITUMINOUS AGGREGATE-1952
6" GRAVEL BASE-1952
12" SUBBASE: SAND
4" ORIGINAL GRAVEL SURFACE
SUBGRADE: SANDY CLAY
DRAINAGE: GOOD
CLASS-II FLEXIBLE
M-57, 1952 SERVICE 6 YEARS
TRAFFIC: TOTAL 1000, COMMERCIAL 200, DHV 130
SURVEY DATES: DEC. 5, 1958, FEB. 20 & MAY 6, 1959

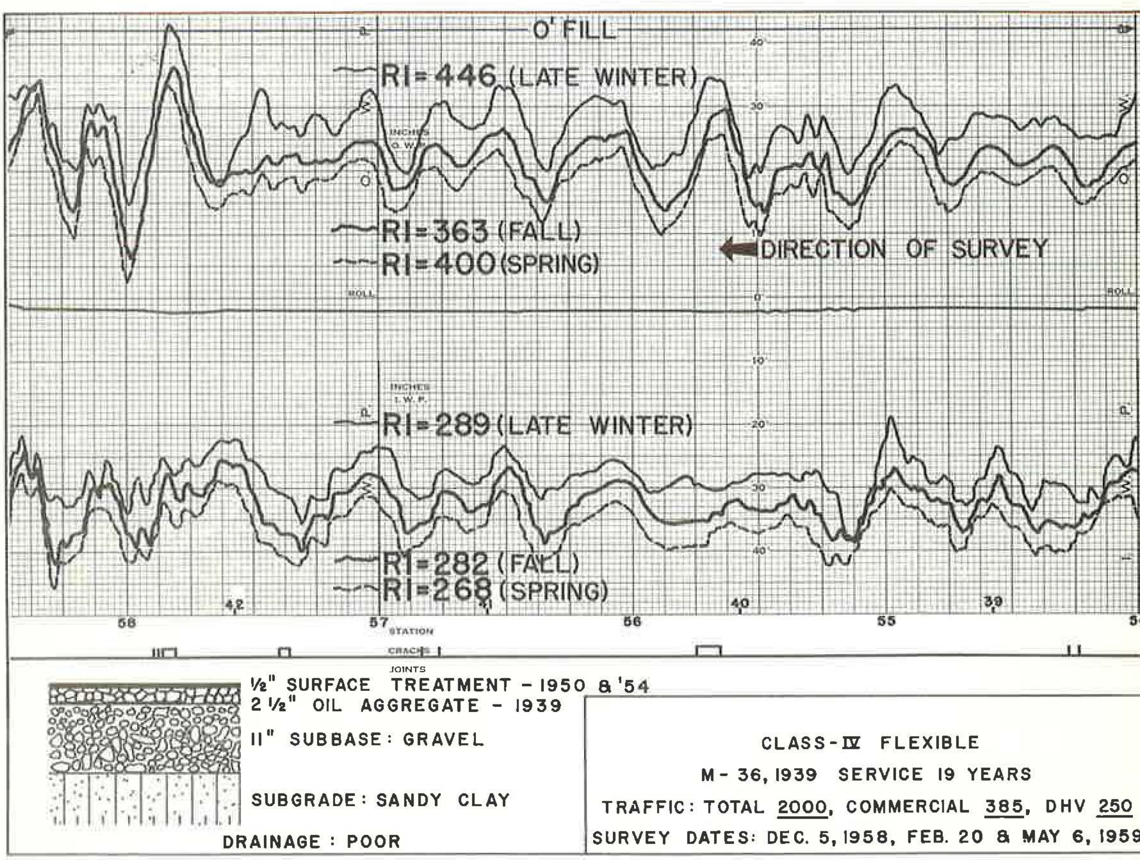
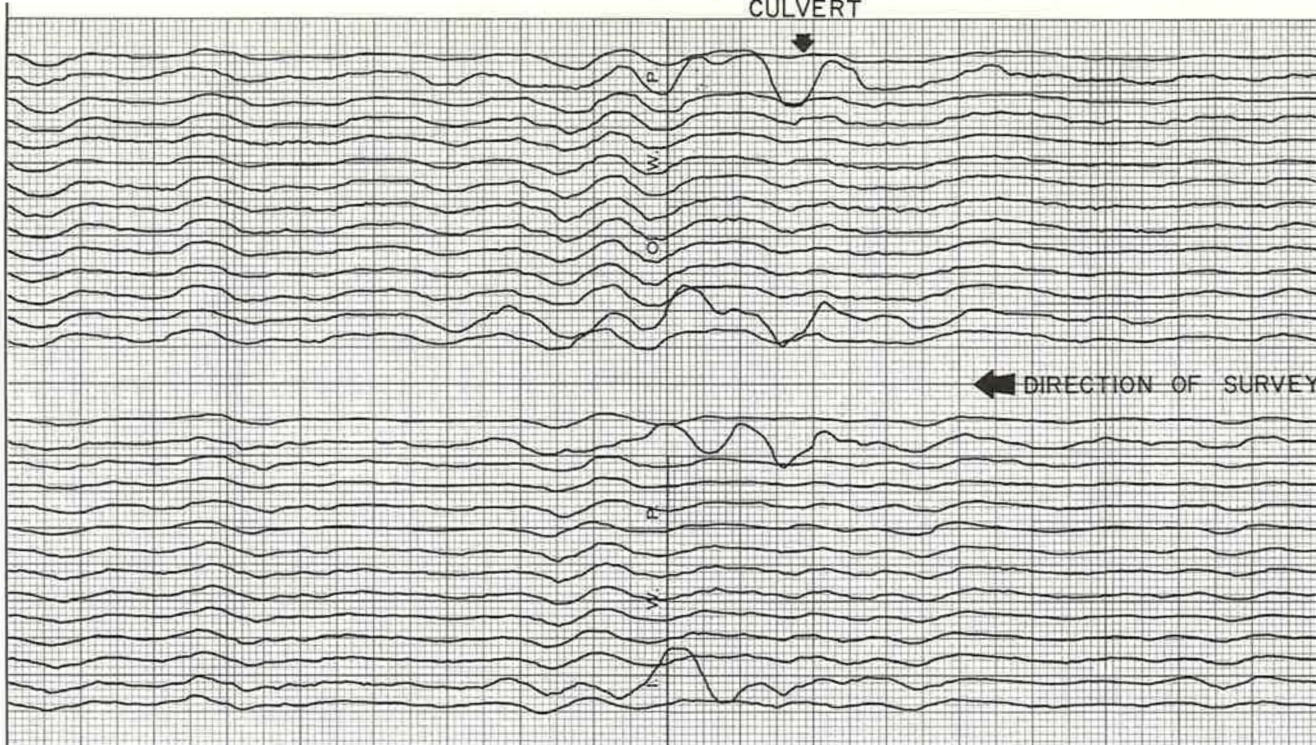
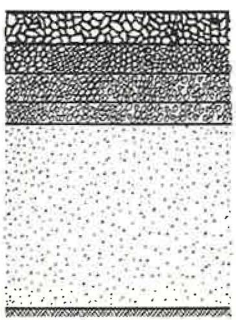

O' FILL
RI=446 (LATE WINTER)
RI=363 (FALL)
RI=400 (SPRING)
DIRECTION OF SURVEY
RI=289 (LATE WINTER)
RI=282 (FALL)
RI=268 (SPRING)
STATION
CRACKS
JOINTS
½" SURFACE TREATMENT - 1950 & '54
2½" OIL AGGREGATE - 1939
11" SUBBASE: GRAVEL
SUBGRADE: SANDY CLAY
DRAINAGE: POOR
CLASS-IV FLEXIBLE
M-36, 1939 SERVICE 19 YEARS
TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
SURVEY DATES: DEC. 5, 1958, FEB. 20 & MAY 6, 1959

Figure 6.



DATE OF SURVEY	ROUGHNESS INDEX (R.I.) IN./MILE
OUTER WHEEL PATH	
9-25-58	33
3-7-59	68
5-21-59	36
10-2-59	43
11-24-59	34
3-8-60	36
9-2-60	41
3-21-61	47
6-21-61	45
9-15-61	36
10-5-61	37
12-27-61	41
3-13-62	68
4-17-62	42
INNER WHEEL PATH	
9-25-58	23
3-7-59	57
5-21-59	26
10-2-59	23
11-24-59	28
3-8-60	27
9-2-60	30
3-21-61	33
6-21-61	37
9-15-61	29
10-5-61	33
12-27-61	34
3-13-62	61
4-17-62	33

THE ABOVE ROUGHNESS INDICES ARE FOR THE PRESENTED SECTION, LOCATED BETWEEN 915' TO 465' SOUTH OF BERG ROAD.



4 1/2" BITUMINOUS CONCRETE
4" AGGREGATE BASE (58% CRUSHED)
4" AGGREGATE BASE (58% CRUSHED) } SPREAD IN ONE
3" SELECTED AGGREGATE SUBBASE } OPERATION

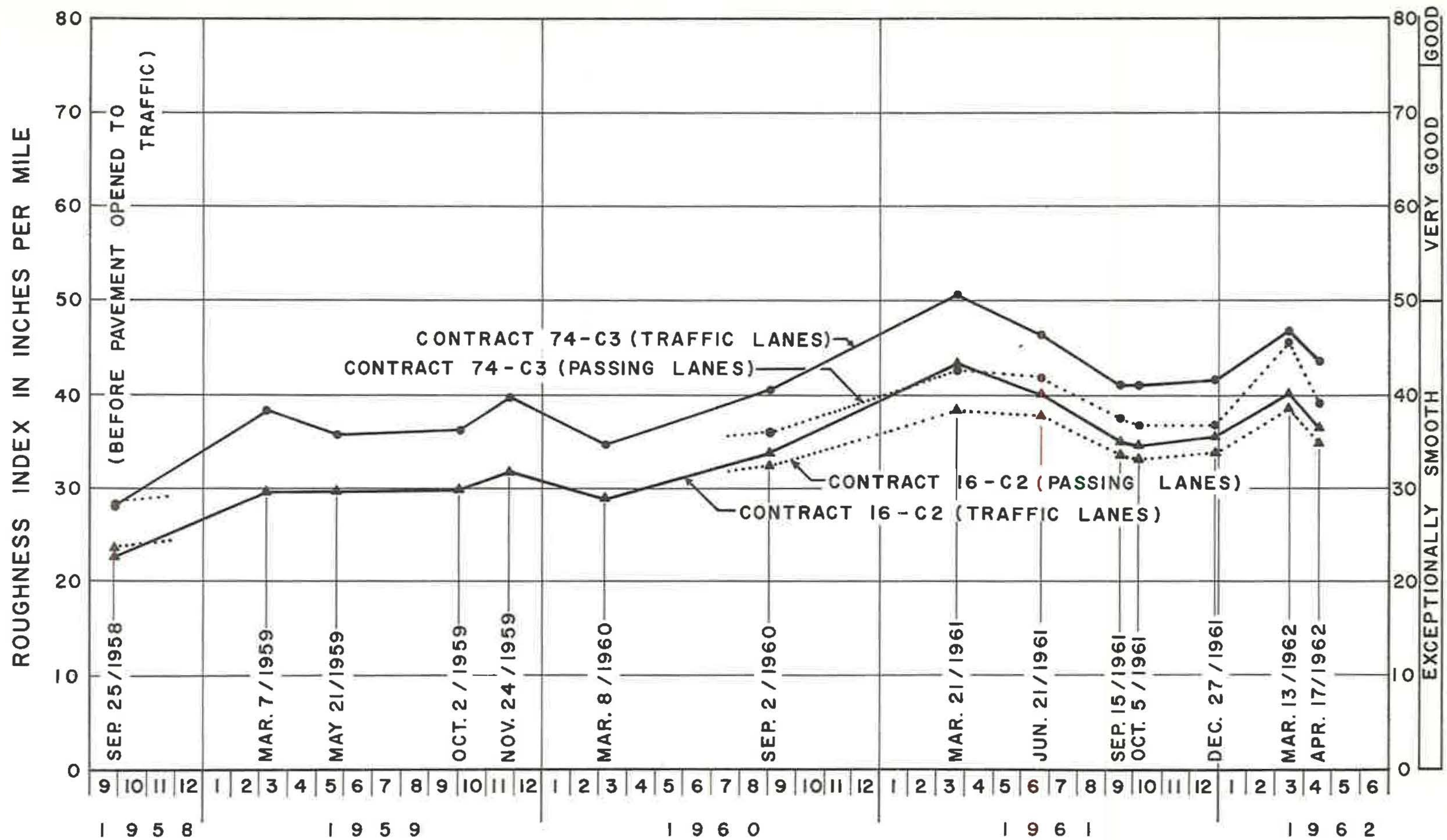
25" (MINIMUM) SAND SUBGRADE

ORIGINAL SOIL - SAUGATUCK SAND

DRAINAGE: EXCELLENT

US-31 MUSKEGON - GRAND HAVEN EXPRESSWAY
CLASS-I FLEXIBLE PAVEMENT
TRAFFIC: TOTAL 8000, COMMERCIAL 1250, DHV 920
PROJECT BM 61074-C3RN, 1958
NORTHBOUND TRAFFIC LANE

Figure 7.



CUMULATIVE CHANGES IN ROUGHNESS

US-31 MUSKEGON - GRAND HAVEN EXPRESSWAY

CLASS-I FLEXIBLE PAVEMENT

Figure 8.

The discussion of the evaluation of pavement performance as related to design may be concluded by summarizing some of the major findings on design correlation during the five-year study.

1. Michigan's current design standards for rigid pavements carrying present legal axle loads are adequate for all-season service without load restriction. In thousands of miles of pavement profile surveys of concrete pavements which by design or natural conditions meet these standards, there has been no significant evidence of loss in serviceability over periods up to 30 yr due to unlimited repetition of legal axle loads. See Figures 3 and 4 with data from the traffic and passing lanes of 244 contracts over 1,275 miles of Class 2 concrete pavement.

2. On the other hand, as shown in Figure 5, concrete pavements that have been designed and built to these standards suffer a cumulative increase in roughness of 4 to 5 in. per mile per year due to climatic and other environmental factors. Chief among these deteriorating influences are the temporary pavement displacements caused by frost action and temperature differentials. Frost displacement appears to originate in the freezing of moisture which accumulates in the subgrade and granular bases and subbases immediately beneath the pavement surface. Temporary displacements, which reach a maximum in late winter, largely disappear in the summer but leave a residual roughness which is the primary source of the cumulative loss in riding quality (5). (See Fig. 6).

3. Flexible pavements with bituminous surfaces built to equivalent all-season standards for present legal axle loads show comparable performance characteristics and evidence of cumulative changes of about the same order of magnitude. The mechanics of flexible pavement are such that cumulative loss of riding quality is not produced by the same type of residual roughness as in rigid pavement; but, on the other hand, there is some evidence of measurable differentials in roughness due to traffic. These considerations and results from short-time studies are inconclusive, although they give some promise that the loss in riding quality may proceed at a lesser rate than in rigid pavements. However, sufficient data over longer periods of service and comparable conditions are still to be accumulated to supplement the present study before these important questions can be answered (4). (See Figs. 7 and 8).

Construction Practice and Pavement Performance

It has been stated that pavement performance surveys have shown that current design standards provide adequate load-supporting capacity. However, these same surveys show that in terms of potential riding quality, the benefits of adequate design are not being fully realized. Involved in this problem are plans and specifications and construction control that fail to achieve the maximum potential performance from well-designed pavements. This appears to fall largely in the field of construction practices and, therefore, is being discussed under that heading. The accumulation of a large volume of pavement profile data has brought to light, or perhaps emphasized by supplying the figures, several deficiencies in construction practice.

Granting that the end product in building a pavement is riding quality, then current specifications and inspection procedures fail to conserve or protect a considerable percentage of a pavement's potential life. Built-in roughness has become a common term only since pavement condition surveys have included accurately recorded profiles and the RI associatied with them. One of the first observations that was somewhat surprising to those unaware of the problem was the sharp contrast between the RI of bridge decks and bridge approaches and that of the adjacent roadway pavements finished with conventional paving equipment. Representative data from departmental and supplementary reports submitted on bridge decks and approaches show RI ranging from about 100 to 300, averaging around 200 in. per mile. In terms of the tentative rating of riding quality, the average performance of bridge decks would be described as very poor to extremely rough. Bridge approaches fall in about the same classification.

Another observation on built-in roughness is the almost universal characteristic of greater roughness in the outside wheel path or the edge of the pavement. This has been taken to indicate that irregularities in form setting were more completely reproduced close to the forms and damped out, to some degree, in the center of the concrete slab.

Turning next to hand-finishing of paving, which occurs in special cases where machine finishing is impossible or has been eliminated by special permission, the results are comparable to those obtained on bridge decks. Supplementary reports were submitted that dealt with the roughness of hand-finished pavement on the ramps of the grade separation at the intersection of M-21 and I-96 near Grand Rapids. The roughness on the first ramp varied from 167 to 191 in. per mile, which would be rated from poor to very poor. The second ramp showed RI varying from 145 to 154, falling on the borderline between acceptable and poor, but certainly not to be considered as high-quality work.

Occasionally some unusual conditions come to light as pavement profiles are being analyzed that may be related to construction methods, as illustrated by the peculiar built-in roughness shown in Figure 2 and discussed previously.

In presenting examples where construction practice has resulted in abnormally high built-in roughness, it would distort the picture to ignore the equally numerous examples where high-grade workmanship has produced superior riding quality. The fact that there are such examples is particularly significant because it demonstrates that it is within the range of common practice in pavement construction to produce such superior results. There is, then, all the more reason why poor workmanship and inferior riding quality need not be accepted.

Several examples of superior riding quality in both concrete and asphalt pavements may be cited. In Figure 5, a group of three concrete pavements built with RI of 50 in. or less per mile are shown; also shown are five other projects which, allowing for normal increase in roughness, would have had built-in roughness of less than 50 in. per mile. It is significant that five of these eight projects were built by two contractors who had established reputations for doing high-quality work (5). Other illuminating examples were also cited in the same report in the discussion of quality of workmanship.

Application of Pavement Profile Data to Maintenance

Data from condition surveys of existing roads are of direct value in several phases of maintenance, with particular reference to the pavement structure. The rate of change in both roughness and structural continuity, when compared with normal cumulative changes, may reflect unfavorable physical conditions or weakness in design and construction that may be possible to correct. Cracking in concrete pavements due to environmental factors or load repetition, or to the combination of both, is a natural development; hence, joint and crack maintenance is accepted as normal and considered a routine operation in the early stages of pavement life. In older pavements, or in those for one reason or another subject to excessive cracking, filling of joints and cracks may become ineffective or prohibitive. Such conditions may be the signal for resurfacing or early reconstruction, beyond the scope of maintenance.

In bituminous pavements, both roughness and loss of structural continuity have significance comparable to those in rigid concrete pavements, but the evidence of structural deterioration is not as easy to evaluate in quantitative terms. Identification and classification of cracking, patching, and other types of surface deterioration in bituminous pavements have been worked out by technical committees of the Highway Research Board and also in connection with the AASHO Road Test. The final reports from that test are perhaps the most readily available and the most authoritative for present use. Consequently, they will be considered in some detail.

In the AASHO Road Test, the RI and continuity ratio used in the Michigan pavement performance surveys are combined in a single numerical index, defined as the present serviceability index, PSI. The Michigan RI and the cracking and patching as a measure of structural continuity in a flexible pavement have been translated into terms of the PSI.

The first step in this procedure is illustrated in a previous report in which the

Michigan RI was converted into a function of the AASHO slope variance, $\sqrt{SV}$, by a theoretical equation developed by Irick (4, Fig. 20). Conversion of comparable data from a number of different projects is shown in Figure 20 as a representative of the general correlation. Figure 21 of the same paper shows, on a semilogarithmic plot, the relationship between the PSI and the Michigan RI derived from the rating of 49 rigid pavements by a panel of observers selected to extend AASHO Road Test results to existing pavements. To test the validity of this relationship, comparative values of both measures of serviceability or performance have been plotted from six flexible and six rigid pavements in Michigan.

The preceding discussion of quantitative measures of pavement performance has two objectives. The first was to show that data from the Michigan Pavement Performance Study can be readily translated into terms of the PSI and, conversely, that useful results from that test could be put into practice in Michigan. The second objective was to apply the pavement performance criteria to maintenance and point out relationships of important practical significance.

Directing attention now to the second objective, it seems particularly important to take note of the fact that deterioration of the pavement surface, reflecting loss in structural integrity, is of primary importance as an independent guide to timely maintenance and should not be buried by the statistical combination involved in reducing pavement performance to a single numerical coefficient such as the PSI. Recognition of this fact has entered into some of the most recent discussion of this subject and it seems reasonable to suppose that pavement performance criteria may be adjusted accordingly.

As a first example of the use of pavement condition data from field surveys as a guide to maintenance, reference is made to a previous paper which is devoted largely to describing maintenance of the airfield pavement at Willow Run (6). Maintenance of the airfield paving was a basic responsibility assumed by the University of Michigan when the University took title to the field in 1946. Although the deed stated "... that the entire landing area ... shall be maintained at all times in good and serviceable condition ...," no standards or procedures were prescribed for judging what would be considered "good and serviceable condition."

This paper outlines the periodic surveys and procedures developed for maintaining a continuous record of pavement condition. Prior to resurfacing, structural continuity, as measured by pavement cracking in terms of the continuity ratio, was the basic measure of pavement condition. Pavement roughness was not a serious problem in the airfield pavement and was not recorded during this period. Joint and crack filling and occasional slab replacement constituted the major part of the maintenance program, and the pavement was never allowed to reach a state of disrepair. As the cracking pattern became more advanced, this type of maintenance became prohibitive and bituminous resurfacing was adopted on an annual incremental program.

After resurfacing, and with the availability of equipment to record pavement profiles and RI, the measure of pavement condition was shifted to cumulative change in roughness, supplemented by visual surveys of reflected cracking. Resealing of the bituminous surfaces before reflected cracking reached an advanced stage was the adopted practice, making timely maintenance the keynote of the program.

From the standpoint of the Michigan study and accumulating experience, it appears desirable to retain both RI and the continuity ratio or its equivalent in evaluating pavement performance, with particular reference to pavement maintenance. Several other examples may be cited which indicate that needed maintenance may frequently be reflected in structural deterioration of the surface well in advance of loss in riding quality. In this connection, it may be noted that fairly substantial amounts of cracking, patching, and rutting have an almost negligible effect in the computation of the PSI (4).

Value of Pavement Performance Data to Operations

In the introductory discussion of the utilization of pavement profile data, the operation of the state highway system as a public facility was set forth as one of the four major functions of a state highway department. Although this may be recognized by highway engineers and administrators, it does not appear to have been given sufficient emphasis as a separate phase of highway responsibility to gain it the public attention its importance deserves. Pavement performance and pavement profile data have to do specifically with the pavement surface itself, the sole purpose of which is to provide superior riding quality for the comfort, convenience, and economic benefit of the highway user.

The Michigan Pavement Performance Study as organized and operated during the five-year period covered by this review provides an excellent example of the value of accurate pavement evaluation in the operation of the highway network to obtain the maximum economic benefit as a statewide transportation facility. One of the major objectives of the sponsors of this project was to provide all-season operation for full legal axle loads and to demonstrate the practicability of such operation by carefully controlled observations of pavement performance.

The first step in this program was the selection of a network of highways on which the spring load limitations could safely be eliminated, and then to expand that network as rapidly as possible. Since 1940, Michigan's design standards for trunkline construction have been gaged to provide all-year service for legal axle loads, without spring load limitation. Consequently, by 1958, a substantial mileage of such roads had been built. The first pavement evaluation, of January 1, 1958 (3, Fig. 1) was prepared as a statewide evaluation of the trunkline system from the standpoint of adequacy to carry legal axle loads without restriction. It included those roads on natural granular subgrades and with natural conditions making them adequate for year-round service (Class 1), and those roads which had been improved with drainage and granular subbases to compensate for seasonal loss of strength (Class 2).

The first pavement evaluation, in 1958, provided an integrated inventory of adequate roads which classified approximately 55 percent of the state trunkline system as adequate for legal axle loads at all times. Based on this evaluation, the first so-called "frost-free" network was established and public notice given of the raising of spring load restrictions on this network as of January 1, 1958. Including additions made as the result of special studies, the unrestricted network during the 1958 "spring breakup" consisted of some 4,545 miles, or about 50 percent of the state trunkline mileage. Judged in terms of public benefit, it was estimated that the cost of spring load restrictions to the state's industry and agriculture was some $20,000,000 per year, of which a substantial part has been saved during the spring season each year since 1958, without significant damage to the roads.

The second phase of the pavement evaluation was the expansion of the unrestricted network as the result of new construction, betterment, and reclassification. The pavement profile surveys entered directly into the reclassification and provided the supporting data to demonstrate that Michigan design standards did provide roads that would not be damaged by legal axle loads under year-round operation. Under this controlled operation of the state trunkline system, the unrestricted mileage has been progressively increased as indicated in Table 3.

TABLE 3

ALL-SEASON TRUNKLINE HIGHWAYS

Date	Length		Reference
	Mi	%	
1 Jan. 58	4,545	49	3 (Fig. 2)
1 Jan. 59	5,519	59	Files only
1 Jan. 60	5,985	64	7 (Fig. 3)
1 Jan. 61	6,344	68	Files only
1 Jan. 62	7,031	76	Files only
1 Jan. 63	7,455	81	Files only

Based on the statewide pavement evaluation, two maps are prepared and issued annually, particularly for the guidance of commercial transportation. These maps are the "All Season Trunkline Highways" and the "Truck Operators' Map." The expansion of the "All Season Trunkline Highways" is graphically illustrated by the annual maps that are issued, which are given in Table 3 with references and the consistently increasing mileage in the unrestricted classification.

The map of "All Season Trunkline Highways" designates the network over which full legal axle loads may be operated at all times. The "Truck Operators' Map" shows a network of highways calculated to provide continuous routes leading to any destination in the state. Not all of these routes are unrestricted in the spring of the year, thus the operators must use the "All Season" map to check loadings.

The "Truck Operators' Map" also shows "Special Tandem Routes" on which a maximum load of 32,000 lb on one set of tandem axles or 16,000 lb per axle is permitted. This loading applies when load restrictions are not in force, including the "All Season" highways at all times. When restrictions are in force, all tandem axles are limited to 26,000 lb or 13,000 lb on each axle.

The publication in January of each year of these two maps represents a permit to truck operators and all other highway users for unrestricted use of the designated routes under the authority of the Michigan State Highway Department. They represent the ultimate result of pavement evaluation of state trunklines in the operation of the state highway system as a transportation facility. What this means in terms of savings to state industry and agriculture has been cited here to illustrate the importance of well-informed operation of a state highway system and the value of pavement performance data in the support of that type of operation.

CONCLUSION

Termination on December 31, 1962, of the last of a series of annual contracts with the Michigan Highway Planning Survey marked the end of the second five-year program of the Michigan Pavement Performance Study. After completion of field surveys of existing pavement in service, providing profiles of 10,000 lane miles of pavement, the Department's concept of a highway pavement has undergone rather substantial change. Some ten years of concentrated study of how pavements react in the field brings realization that pavement performance cannot be measured in terms of static equilibrium of a beam resting on an elastic foundation subjected to static loads with strength controlled by a direct proportionality between load, deflection, and thickness.

On the contrary, an objective viewpoint sees the pavement slab expanding and contracting with changes in temperature; curling and warping with temperature differentials between the top and bottom; growing and shrinking with moisture changes; and distorted by frost displacement, only partially relieved by thawing of the frozen substructure. All of these effects superimposed on stresses due to load make the life of a pavement an everchanging cycle of dynamic effects, which seems to require a new and more realistic concept of pavement performance and poses another set of questions that was new ten years ago. How does an adequate pavement react to these changing conditions? How long does it retain an acceptable riding quality? What is normal behavior, in terms of which abnormal behavior can be identified and defined?

It is hoped that the Michigan Pavement Performance Study has provided some answers to these questions, in terms of which the responsible factors that control pavement performance can be isolated and logical relations between cause and effect may be determined.

In conclusion, it is even more difficult but necessary to restate in concise form the principal conclusions drawn from the study, as follows:

1. Pavement performance has been evaluated in terms of two basic measures of the physical condition of the pavement defined as RI and the continuity ratio. Both of these quantities are required to evaluate the pavement condition at any given time. The RI, in conjunction with the recorded pavement profile, measures the riding quality or serviceability of the pavement; the pavement profile supplies an insight into the source of the progressive changes which have taken place during the life of the pavement. The continuity ratio expresses in quantitative terms the structural continuity of the slab and enables one to anticipate its ability to continue in service without excessive deterioration due to load application. It indicates the need for maintenance or improvements to forestall excessive loss of riding quality.

2. The extensive mileage of recorded pavement profile and supplementary data constitute an accurate and realistic pavement inventory, the value of which has been only partially utilized to date. Its full value to design and construction practice and in the

operation of the state highway system as a transportation facility can be realized only by its continued use and by keeping it up-to-date and growing as the highway system grows.

3. Michigan's current design standards for rigid, portland cement concrete pavements are adequate for legal axle loads, providing all-season service without load restriction. This conclusion is supported by the fact that pavement profile surveys of thousands of miles of such pavement on natural or modified subgrades meeting those standards showed no significant loss of riding quality due to unlimited load application over service periods up to 30 yr.

4. Somewhat in contrast, these surveys provided evidence that concrete pavements deteriorate due to climatic and other environmental influence, losing riding quality in terms of roughness at an average rate of 4 to 5 in. per mile per year. Assuming RI of 200 to 250 in. per mile as the limit of acceptable riding quality, an initial RI of 50 would set the useful pavement life at 30 to 40 yr until resurfacing or reconstruction would be required.

5. Flexible pavements with bituminous surfaces built to equivalent all-season standards show comparable performance. Such flexible pavements also suffer a cumulative loss of riding quality of comparable magnitude even though the mechanics of flexible pavement produce a quite different relation between cause and effect. Conclusions concerning the performance of flexible pavements must be qualified in the Michigan study by the fact that present profile data are limited both in mileage and periods of service.

6. With full realization that pavement life and serviceability are controlled more by environmental effects than by load application, pavement design practice may be pointed in the future more directly toward compensating for these natural destructive influences. The range of pavement performance covered by the present profile surveys is sufficiently large and the contrast between the best and poorest performance such as to indicate that emphasis in design on these environmental factors may produce substantial improvements.

7. Pavement profile data have produced much evidence that pavement construction practice can be improved by more attention to riding quality produced and to those questionable practices which are the primary source of poor riding quality. Initial roughness built into the pavement presently takes up too much of the range available to absorb the cumulative roughness over the years. This may be reflected directly in a reduced useful life of a pavement.

8. Pavement profile surveys and the two factors for evaluating pavement condition, the RI and the continuity ratio, provide reliable and accurate criteria for gaging serviceability and determining when and what maintenance should be provided. To perform this function effectively, profile data as a pavement inventory should be kept up-to-date and these records made readily available to those responsible for maintenance. The development of cracking, as a measure of structural continuity, and other direct evidence of structural deterioration are necessary and timely indications of needed maintenance which anticipates loss of riding quality.

9. A complete and accurate inventory of the state highway system has direct value in several ways in the operation of the highway system as a transportation facility. It provides a factual basis for eliminating unnecessary restrictions on the use of the highways, with economic benefits exceeding many times the cost of providing and maintaining that inventory. It provides the basis for extending the unrestricted network of state highways and the evidence that determines whether or not the continuation of unrestricted use is justified. In this time when pavement design is on trial all over the country, it provides realistic and incontrovertible evidence of the soundness of Michigan design standards and points the way to further improvement in carrying out these standards under varying field conditions.

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