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Evaluation of Pavements by Deflection Studies for Maintenance Purposes

4 Reports

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Foreword

Maintenance engineers use overlays of existing flexible pavements to meet various service demands of these driving surfaces. When an overlay is used to strengthen an existing pavement, two questions are asked: what is the present load-carrying capacity of the existing road; and how thick must the overlay be to bring this road up to a desired load-carrying capacity?

To help the maintenance engineer answer these questions, a session at the 45th Annual Meeting was built around the theme, "Evaluation of Pavements by Deflection Studies for Maintenance Needs." Deflections under moving loads are good predictors of pavement performance. Consequently they can be used as a convenient tool to record what the road says it wants and needs in overlay construction.

Three papers in this Record show how Benkelman beam deflection studies can help the maintenance engineer evaluate the structural conditions of pavements, and how they can be used in the solution of the maintenance problem of rehabilitation. They come from three widely separated areas, namely, Canada, California and Brazil.

The Canadian Good Roads Association has made extensive load bearing evaluations of pavements as measured by the elastic rebound Benkelman beam deflection test. Huculak presents a good picture of the method of rehabilitation of flexible pavements in Canada utilizing deflection studies. He presents a very comprehensive and interesting highway category and action chart. It is a summary of the developments in the field of pavement evaluation by surface deflections and observed performance. This chart is an aid in establishing the maintenance work to be done on a specific road.

The California Division of Highways has long recognized the importance of pavement deflections. For the last six years they have made deflection studies utilizing either the traveling deflectometer or Benkelman beam. Their purpose was to determine the extent of required maintenance of the roadways under investigation. Zube and Forsyth summarize the experience on several individual deflection studies of an unusual nature. They examined in detail the relationship of deflection data and deflection measurements before and after the overlay.

It is of interest to note the similarity in the use of deflection studies in Brazil compared with Canadian and California applications. Carneiro presents a well-conceived long-range plan for the use of deflection data in flexible pavement design and rehabilitation. The question of tolerable deflection levels is given particular attention.

Another paper outlines the Dynaflect equipment which measures deflections of flexible pavements under a moving load. Scrivner and Swift report data which show a satisfactory correlation between deflections obtained with the Dynaflect and with the Benkelman beam. They also report on deflection information obtained by use of Dynaflect over a number of Texas highways.

Benkelman's discussion of the four papers establishes that there is general agreement on the ability of overlays to reduce deflections. The degree of reduction depends upon the overlay thickness. He points out that there is a diversity of opinion on the question of tolerable deflection limits which should be obtained by the overlays. They vary with traffic, environment and conditions of the project under construction. Benkelman sets forth certain guideline values for those who are contemplating the use of deflection data in pavement rehabilitation work.

The papers in this Record provide procedures for taking maintenance out of the "hit or miss" technique. Here are presented data for the development of a recommendation for overlay thickness of flexible pavements by the deflection method.

Contents

A New Research Tool for Measuring Pavement Deflection

F. H. SCRIVNER, Research Engineer, Texas Transportation Institute, Texas A & M University;
GILBERT SWIFT, Manager, Special Projects, Lane-Wells Company, Houston, Texas; and
W. M. MOORE, Assistant Research Engineer, Texas Transportation Institute, Texas A & M University

•IN CONNECTION with current research being conducted for the Texas Highway Department and the U. S. Bureau of Public Roads, it became necessary in 1964 for the Texas Transportation Institute to measure deflections on several hundred flexible pavement sections on highways throughout Texas. Before initiating a program of that size, we decided to investigate a device recently developed by Lane-Wells Division of Dresser Industries, Inc., capable of recording the deflection of a road surface caused by the application of a relatively light oscillating load. If it could be shown that the deflection so induced correlated reasonably well with static deflection measured by conventional means, we felt that certain unique advantages of the device would warrant its use in our research.

This report describes the Lane-Wells measurement system, gives the results of the preliminary investigation, and presents data illustrating how the deflection basin is ...cted by variations in the structural design of the pavement. It also describes an ...proved model of the system developed in 1965 by Lane-Wells as a result of their experience in this research.

Figure 1. Dynaflect halted for measuring roadway deflections; force generator is located in covered trailer; deflection pickups rest on pavement beneath trailer tongue.

1965 VERSION OF DEFLECTION MEASURING SYSTEM

General Description

The Lane-Wells Dynaflect, developed after completion of the 1964 measurements program on Texas highways, consists of a small two-wheel trailer containing a dynamic force generator and equipped with a set of motion-sensing devices (Fig. 1).

Deflections of the roadway, or other material beneath the trailer, caused by a cyclic downward force, are measured while the trailer is halted briefly at each test location. Deflections are read directly on the meter shown in Figure 2.

Dynamic Force Generator

The cyclic force is produced by a pair of counter-rotating unbalanced flywheels

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Paper sponsored by Committee on Maintenance of Bituminous Pavements and presented at the 45th Annual Meeting.

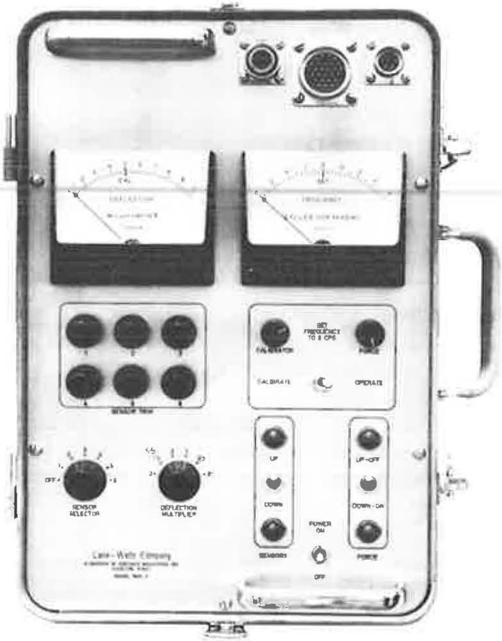


Figure 2. Control panel with frequency and deflection meters.

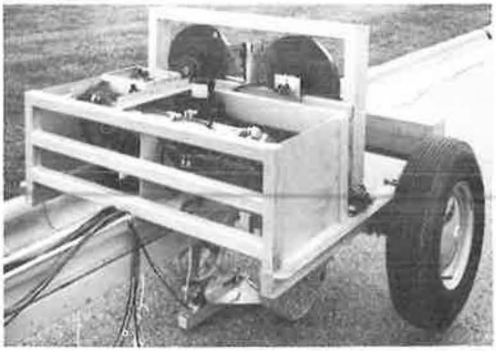


Figure 3. Trailer with hood removed; force generated by unbalanced flywheels at top of picture is transmitted to road through steel wheels at bottom.

(Fig. 3). Two eccentric rotating masses produce a vertical reaction force which is transmitted to the ground. The horizontal reactions cancel by virtue of the opposing rotations. The instantaneous force is proportional to the unbalanced mass and to its vertical acceleration. Accordingly, its value is given by the expression:

$$F = 4\pi^2 f^2 r \frac{W}{g} \sin (2 \pi f t)$$

where

F = force, in lb;
f = rotation rate, in cps;
r = radius of eccentric mass, in ft;
w = weight of eccentric weight, in lb;
g = acceleration of gravity, in ft/sec^2;
t = time, in sec.

In the present model, operating at 8 cps, the dynamic force varies in sine-wave fashion from 500 lb downward to 500 lb upward during each rotation, a total excursion of 1,000 lb. The entire force applied to the ground consists of the weight of the trailer,

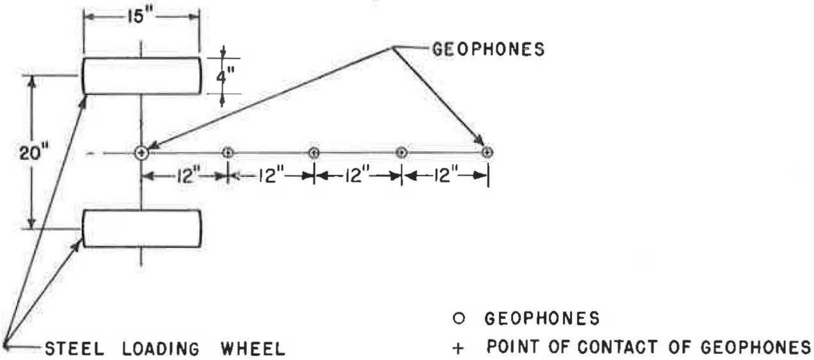


Figure 4. Typical arrangement of geophones used with the Dynaflect (plan view).

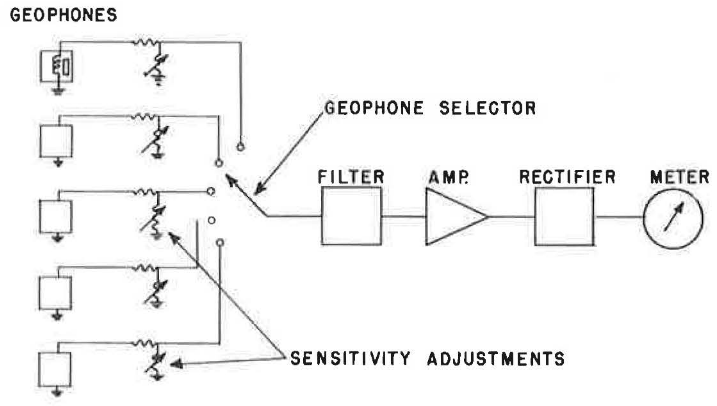


Figure 5. Motion sensing and measuring system.

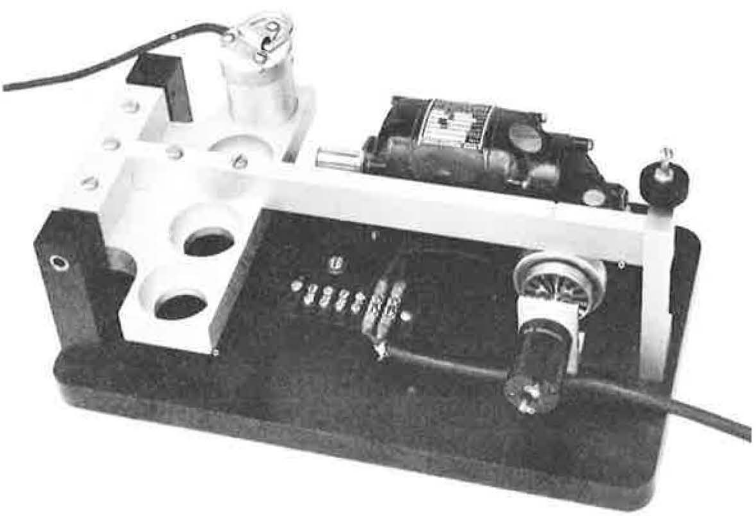


Figure 6. Device for calibrating geophones.

approximately 1,600 lb, together with the dynamic force which alternately adds to and subtracts from the static weight. Since the static weight amply exceeds the dynamic force, there is always a substantial downward component. Thus there is no tendency for the device to lose contact with the ground.

The cyclically varying force is applied to the ground through a pair of steel wheels, whereby the equipment can be moved readily from one measuring point to another.

The rotation rate of 8 cps has been chosen sufficiently low to provide good correlation with static deflection measurements, yet sufficiently high to render the apparatus simple and compact.

Deflection Measuring System

The material to which the dynamic force is applied deflects in synchronism with the force, not only directly beneath the wheels, but throughout a nearby region which constitutes the deflection basin. The amplitude of this induced cyclic vertical displacement is sensed by geophones (seismometers) which are lowered into contact with the

Figure 7. Dynaflect is controlled and deflections recorded inside towing vehicle.

surface at appropriate distances, between a few inches and several feet, away from the steel wheels (Fig. 4). Because these distances are all large in comparison with the area of contact between the wheels and the surface, variations of the contact area have negligible effect on the observed deflections.

The geophones respond to the 8-cps induced motion and produce electrical signals proportional to this motion. Since the displacements are cyclic, their measurement does not require a fixed reference point in the vicinity. For this reason, the dynamic method is immune to the errors encountered with other methods when their reference points lie within the deflection basin.

Each geophone consists of a coil, spring-suspended for vertical motion, within the field of a permanent magnet. When the magnet is subjected to cyclic vertical motion, the coil tends to remain stationary. Accordingly, the coil acquires a cyclic velocity with respect to the magnet and a voltage proportional to the instantaneous velocity is developed within the coil. At any single frequency of excitation, the magnitude of the geophone output voltage is precisely proportional to its motion.

The geophones are used, one at a time, to determine the deflection at each point in the array (Fig. 5). The electrical output signal from each geophone is filtered and amplified to produce a reading on a meter. The narrow-band filter limits the response of the system to the fundamental frequency component of the induced motion at 8 cps. Thus the meter readings represent only the displacements induced by the force generator and are unaffected by extraneous vibrations caused by moving traffic or other sources. Deflections up to a maximum of 30 thousandths of an inch and down to a minimum of 0.01 thousandth can be measured with the present apparatus.

Standardization of the deflection measuring system is accomplished by placing each geophone on a cam-adjusted platform which provides a smooth, repetitive, 0.005-in. vertical motion at 8 cps (Fig. 6). Individual sensitivity controls associated with each geophone are then adjusted to obtain the corresponding reading of 5 milli-in. on the deflection-indicating meter.

Operational Characteristics

A lift mechanism in the trailer moves the force generator in or out of contact with the ground. When lifted, the trailer is supported on rubber tires for travel at legal driving speeds. With the force generator in contact, the unit may be moved on its steel wheels from one measuring point to another at speeds below 10 mph. To enable such moves to be made rapidly, the geophones are raised and lowered by remote control (Fig. 7).

Setup and calibration requires less than 3 min. Measurement of the deflections at each location takes less than 1 min.

1964 VERSION OF DEFLECTION MEASURING SYSTEM

The deflections reported herein were made with a system (Fig. 8) that differed significantly from that described previously only in the following respects:

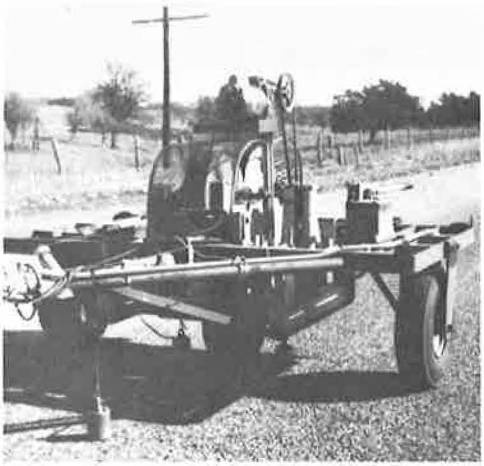


Figure 8. 1964 model of Dynaflect which was correlated with static deflections on Texas highways.

Figure 9. 1964 model in which force was transmitted to roadway through a single steel wheel.

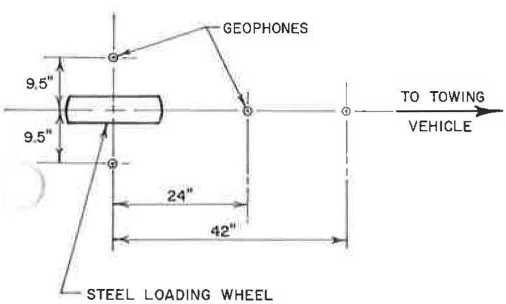


Figure 10. Arrangement of geophones in relation to point of load application used in correlation study and in routine measurements made in 1964; the average output of the two geophones at 9.5 in. from load was correlated with Benkelman beam deflections.

Figure 11. Three Benkelman beams sometimes used in 1964 correlation study: beam in background measured deflection at wheel load; two beams in foreground registered movement at the supports of distant beam.

1. The rotation rate in the 1964 model was 7. 1 cps (instead of 8 cps).

2. The dynamic force varied sinusoidally from 242 lb upward to 242 lb downward (instead of 500 lb upward and 500 lb downward).

3. The force (Fig. 9) was applied to the ground through a single steel wheel (instead of through two wheels).

4. The geophones were placed at distances of 9. 5, 24 and 42 in. from the load as shown in Figure 10 (instead of as shown in Fig. 4).

5. The readout device was a trace made by a pen on moving paper tape (instead of a direct reading meter).

INVESTIGATION OF 1964 SYSTEM

The investigation of the 1964 equipment took the form of a correlation study of the output of the system with the rebound deflection of a 9,000-lb wheel load as measured h a Benkelman beam. The procedure was as follows.

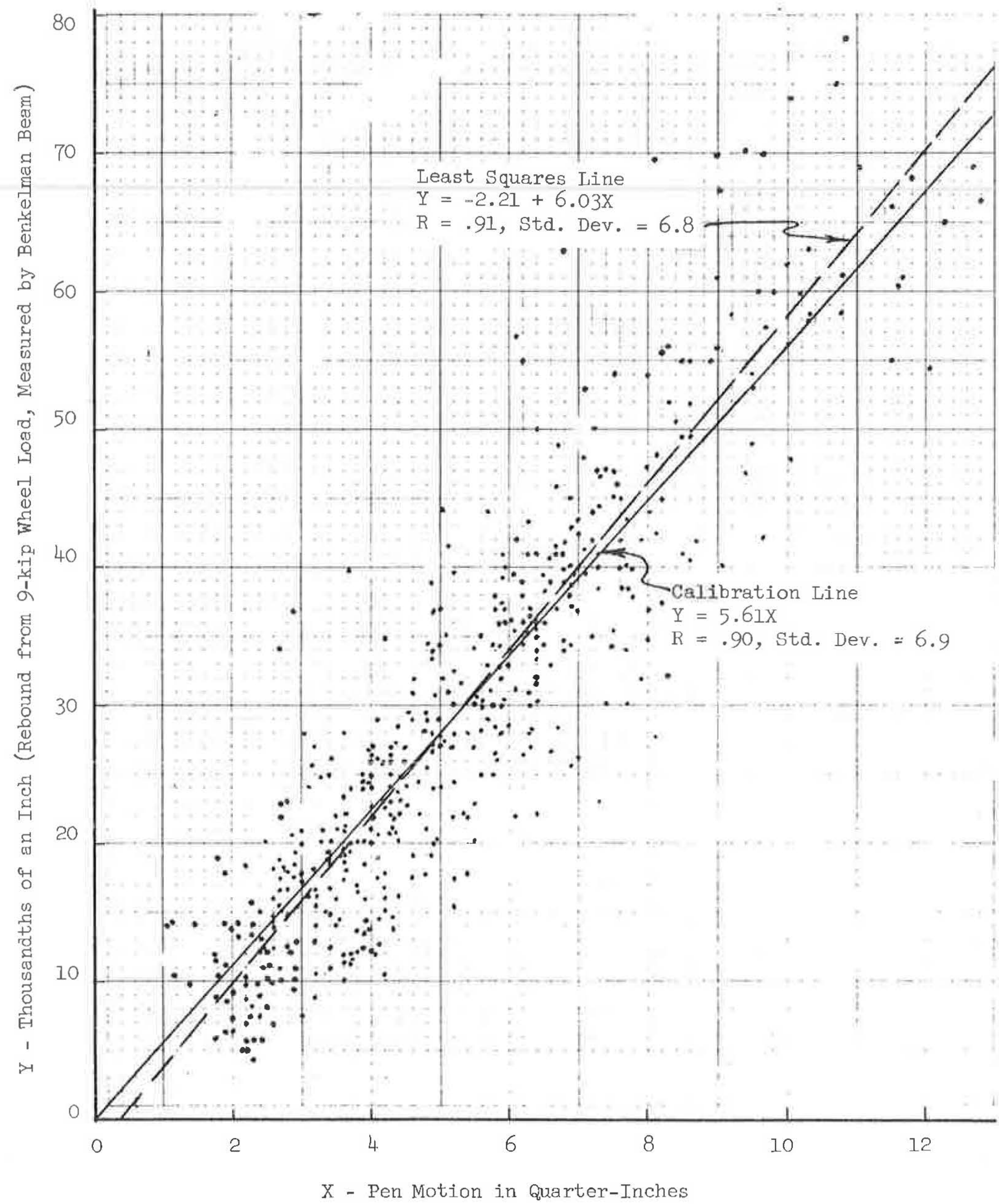


Figure 12. Calibration data for 1964 equipment: each of the 490 points represents a direct comparison between a reading taken at a point on a flexible pavement and a Benkelman beam measurement made at the same point (about 5 min later) of the rebound, following removal of a 9-kip wheel load.

A point in the outer wheelpath of a flexible pavement was tested with the dynamic deflection system. A keel mark was made at the point where the oscillating load was applied. The instrument van and trailer were then driven ahead and a heavy truck, with a load of 18,000 lb on the rear axle, was placed with the center of its outer dual wheel directly over the previously marked point. The probe of a Benkelman beam was then placed on the mark between the outer wheels, and an initial reading of the dial was recorded. The truck was then driven ahead about 50 ft, and a second readin

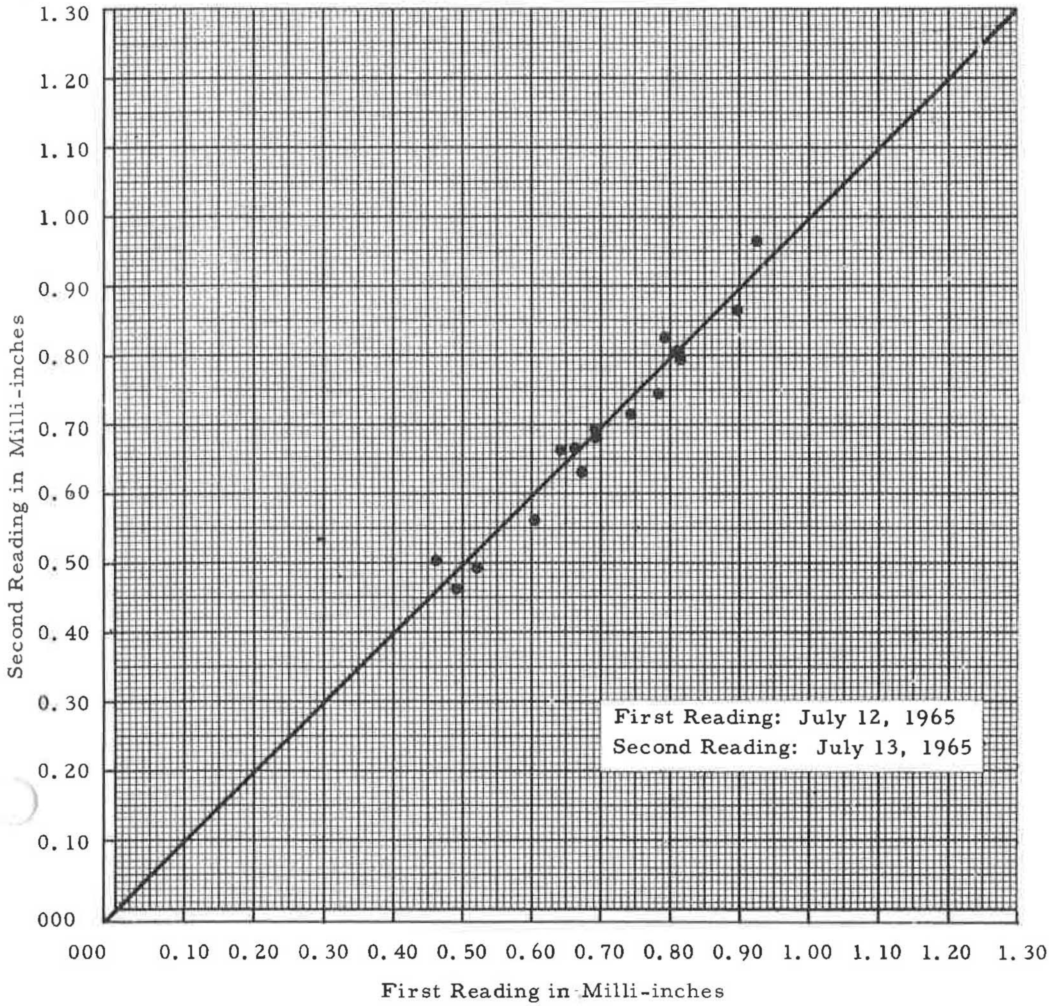


Figure 13. Repeatability of Dynaflect measurements.

of the dial was taken. From the two readings, the rebound deflection was calculated and recorded.

The truck was again placed on the mark and a second Benkelman beam rebound deflection was measured in the manner previously described. The two deflections were averaged and recorded as the Benkelman beam deflection at the point.

Some of the earlier Benkelman beam data, when compared with the Lane-Wells measurements, indicated that the front support of the Benkelman beam may have been within the deflection basin in a few cases. For this reason, a second beam was frequently used to check movement of the front support (sometimes at the rear support, also), and where movement was found the Benkelman beam data were corrected accordingly (Fig. 11). Except in the case of exceptionally stiff (stabilized) pavements, however, the observed movements of the supports of the Benkelman beam were small, and the force of the wind acting on the instrument frequently made the reading of these motions difficult and unreliable. As a result, the greater portion of the Benkelman data were not corrected for movement of the supports.

Comparison of the reading recorded by the dynamic deflection system with the Benkelman beam system was made at fourteen points in the outer wheelpath on each

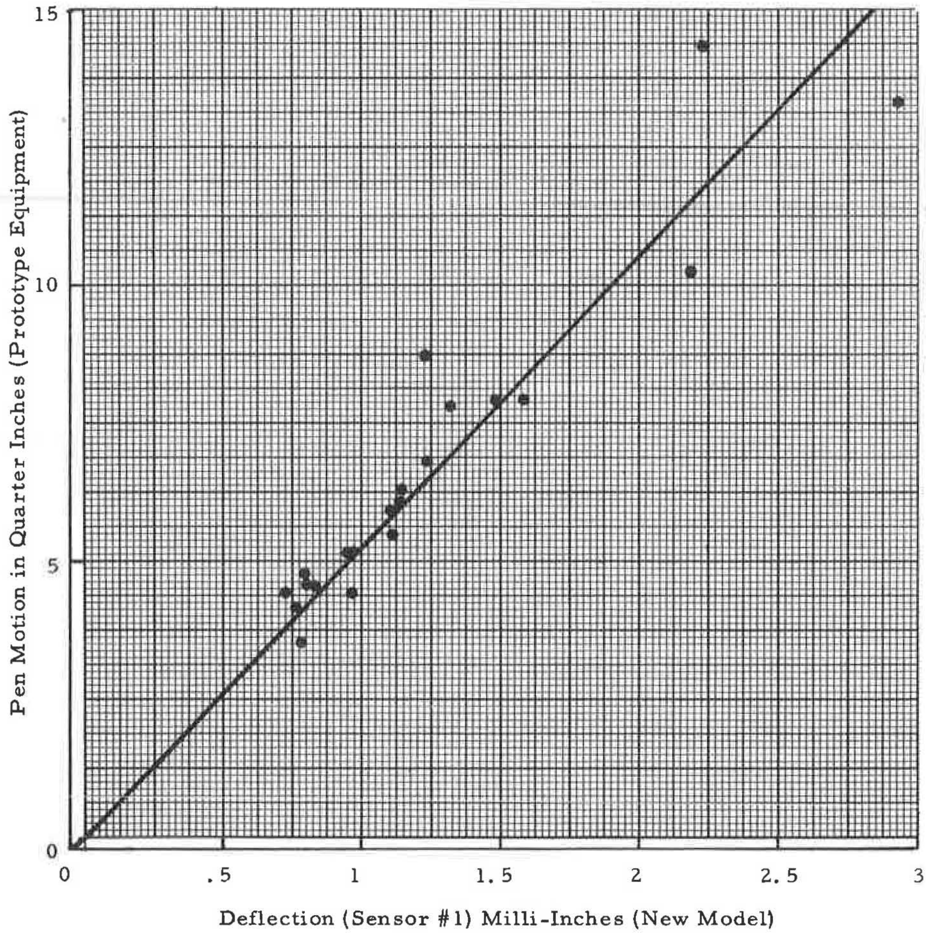


Figure 14. New model Dynaflect compared with prototype.

of thirty-five flexible pavement test sections, twelve of which were in Texas Highway Department District 12 (near Houston) and the remaining twenty-three in District 9 (near Waco).

Figure 12 shows readings of the Lane-Wells device representing the average of two geophones 9.5 in. from the oscillating load, plotted against the rebound (measured by the Benkelman beam) of the pavement surface at the same location after application and subsequent removal of a 9,000-lb wheel load.

There were 490 data points available for analysis. A least-squares regression yielded a correlation coefficient of 0.91 between the two tests, and indicated that a Benkelman beam deflection could be predicted from a Lane-Wells test with a standard deviation of 0.007 in. Figure 12 shows that the line fitted by minimizing the squared errors intercepted the Y (Benkelman beam deflection) axis at -0.0022 in. and had a slope of 6.03. A second line passing through the origin of the graph, and with a slope so chosen that the sum of the deviations of the data points from the line would be zero, has the equation

$$Y = 5.6X$$

where

Y = Benkelman beam deflection in thousandths of an inch, and
X = reading of dynamic deflection recording pen.

Although the least-squares line is perhaps slightly more accurate, the second line is more convenient for use in converting the dynamic deflection data (for the geophone located 9. 5 in. from the load) to estimates of deflections that would be caused by a 9,000-lb wheel load.

CORRELATION STUDY

Dynamic deflections measured at the same location on two successive days have been found to repeat within close limits. The results of one such comparison on flexible pavements are shown in Figure 13.

Some of the scatter of the data evident from Figure 12 may have resulted from the fact that the dynamic deflections were affected by the inertia of the pavement structure, whereas the static deflections were not. A simple example of this source of scatter would be the case of the two pavement structures equal in all linear dimensions and mechanical properties except density. These two pavements would logically deflect equally under a static load but unequally under a dynamic load. Deflection data from two such pavements, if plotted on a diagram like Figure 12, would appear as two separate points on the same horizontal line.

Another source of scatter may have been the fact that dynamic deflections were measured at a point located 9. 5 in. from the load, whereas static deflections were measured at the center of gravity of the load. Thus, two pavements exhibiting equal static deflections would yield different dynamic deflections if the two deflection basins differed in shape. As in the case discussed in the previous paragraph, deflection data from these two pavements would appear as two points on a horizontal line in Figure 12.

Regardless of the reasons for the scatter of the data, the following conclusions are drawn from the correlation study:

1. Although considerable scatter of the data is evident from Figure 12, the relatively high correlation coefficient was taken as good evidence that the 1964 Lane-Wells system responded to those properties of a flexible pavement structure that govern the deflection of the pavement under moving wheel loads.
2. The 1964 device appeared to be rugged, rapid, reliable, and more economical to operate than other systems known to the writers, especially in cases where the objective is to determine the shape of the deflection basin.
3. Based on the foregoing, the decision was made to use the instrument in our 1964 testing program involving several hundred flexible pavement sections.
4. While the correlation study was confined to the 1964 system, the authors are confident that the improved 1965 equipment would correlate as well, and possibly better, with static deflections, as the same design principles were used, and the distance from the load to the nearest geophone was reduced from 9.5 to about 2 in. (The conversion factor determined for the 1964 equipment for estimating 9,000-lb wheel load deflections, however, does not apply to the new equipment because the load configuration is different.) A comparison of the deflections measured by the two pieces of equipment at the same locations on flexible pavements is shown in Figure 14.

EXAMPLES OF MEASURED DEFLECTION BASINS

Existing Highways

Figure 15 shows the approximate range of the deflection determined in the 1964 program, during which 376 sections were tested, and illustrates the general shape of the deflection basins found on existing highways.

Each curve in Figure 15 represents an average of the deflection basins determined at fifteen locations in the outer wheelpath of a 2,500-ft test section. Design data for the five sections, including the Texas Highway Department's Triaxial Class for base, subbase (when present) and subgrade, are given in Table 1.

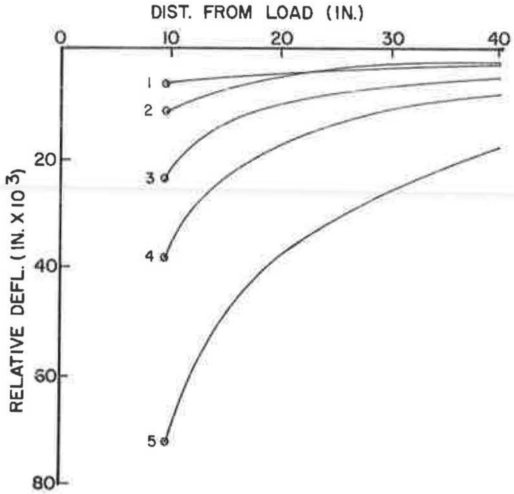


Figure 15. Typical deflection basins measured on Texas highways; circled points represent estimated static deflection under a 9,000-lb wheel load (see Table 1 for design data).

The deflection data for plotting the basins in Figure 15 were obtained by multiplying the output of the geophones (located at 9.5, 24, and 42 in. from the point of load application) by the calibration factor obtained in the correlation study previously described. Thus, the circled points plotted on the graph represent the estimated static deflection that would be caused by application of a 9,000-lb wheel load. It was felt that this method of presenting the data would be more meaningful than showing the actual deflections gaged by the oscillating load.

Special Sections

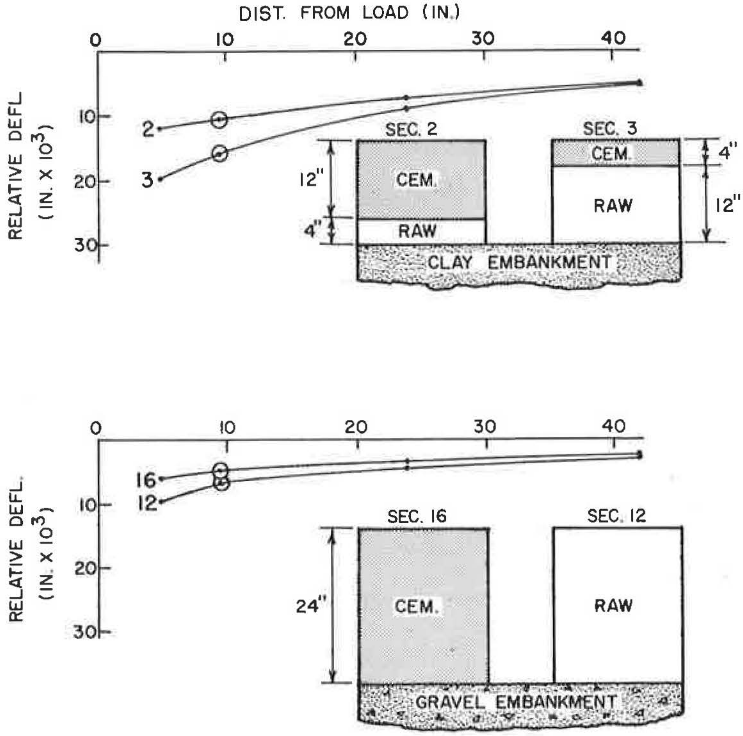
Figure 16 is similar to Figure 15 except that the data were obtained on newly constructed test sections before surfacing, and an additional geophone, placed at a distance of 5 in. from the load, was used. The four sections represented in the figure are a part of a statistically designed experimental facility recently constructed at Texas A & M University to provide a means for evaluating nondestructive pavement testing systems of all kinds. Sponsors of the facility are the Texas Highway Department, the U. S. Bureau of Public Roads, and the Highway Research Board in its role as administrator of the National Cooperative Highway Research Program.

The upper graph of Figure 16 shows the effect on the deflection basin of increasir. the proportion of cement-treated to untreated crushed limestone in a 16-in. structure resting on a clay subgrade. In the lower graph the deflection basin of a 24-in. layer

TABLE 1

DESIGN DATA OF HIGHWAY PAVEMENTS FOR WHICH DEFLECTION BASINS ARE SHOWN IN FIGURE 15

Curve No.	Layer No.	Material	Thickness (in.)	THD Class	Est. Static Defl. (in. $\times 10^3$)	Test Section No.
1	1	Hot mix A. C.	7.0	—	6.5	521-4-2
	2	Cr. limestone	15.3	1.0		
	3	Clay	—			
			22.3			
2	1	Surf. treatment	0.5	—	11.8	244-2-1
	2	Iron ore gravel	9.0	1.0		
	3	Sandy loam	8.0	2.5		
	4	Sandy clay	—	3.5		
			17.5			
3	1	Hot mix A. C.	3.3	—	24.0	15-14-1
	2	Cr. limestone	15.4	2.0		
	3	Clay	—	5.2		
			18.7			
4	1	Cold mix A. C.	0.9	—	38.7	20-1-1
	2	Caliche	8.4	2.8		
	3	Plastic caliche	—	3.8		
			9.3			
5	1	Surf. treatment	0.8	—	72.1	220-7-1
	2	Caliche	3.5	2.0		
	3	Sandy clay	7.5	2.5		
	4	Clay	—	4.0		
			11.8			



MATERIALS CODE: CEM. = CEMENT TREATED CRUSHED LIMESTONE
RAW = UNTREATED CRUSHED LIMESTONE

Figure 16. Deflection basins from four geophones on new sections before surfacing (designs correspond to curves); deflection scale refers to static deflection under a 9,000-lb wheel load as estimated from geophone located 9.5 in. from load (see circled points).

of cement-treated limestone on a gravel subgrade is compared with the somewhat deeper basin of an equal thickness of untreated limestone on the same subgrade.

At the present writing (August 1965) construction of the test facility mentioned previously is nearing completion. We expect later to make many other comparisons of the type described and to relate the size and shape of the deflection basin to the design of the test section. Such a relationship is expected to be useful in the interpretation of deflection data gathered on Texas highways.

CONCLUSIONS

Experience to date in the use of the dynamic deflection system described herein warrants the conclusion that it is rugged, reliable, simple and economical in operations, and capable of becoming a useful tool in pavement research concerned with the measurement of the relative stiffness of pavement structures.

Evaluation of Pavements to Determine Maintenance Requirements

N. A. HUCULAK, District Engineer, Department of Public Works of Canada, Calgary, Alberta

Contrary to normal practice employed in the design and construction of a modern highway, the maintenance aspects of providing this facility are seldom based on adequate pre-engineering and evaluation data. Sufficient time is seldom available to perform a thorough diagnosis of the maintenance problem through conventional sampling and testing techniques. A method of pavement evaluation based on surface deflections and observed performance of existing routes is described and summarized in the form of charts which permit a ready appraisal of the problem as an aid in the establishment of maintenance warrants and highway planning.

•FOR THE past several years, the Committee on Pavement Design and Evaluation has been engaged in a cooperative investigation program to develop economical methods of design, strength evaluation and serviceability of rigid and flexible pavements suitable for Canadian conditions of environment and traffic. Generally speaking, our deliberations have been centered on obtaining an inventory of existing pavements and studying their structural serviceability and performance with relation to age and season. Mo recently, our efforts have been drawn to a more detailed study of the numerous variables affecting this performance.

As one would expect, a considerable volume of data has been gathered, some of which has been interpreted and reported during previous sessions and other publications. Methods of design and strength evaluation have been developed and are being applied extensively across the country. Although the detail analysis is still in progress, experience, coupled with knowledge already gained, permits application of the basic approach on an even wider scale.

Contrary to normal practice employed in the design and construction of a modern highway, the maintenance aspects of providing this facility are seldom based on adequate pre-engineering and evaluation data. In most cases, this is not due to a lack of appreciation of the need for these considerations, but rather the circumstances that surround the maintenance problem. Maintenance requirements must be established shortly after the problem occurs. Sufficient time is seldom available to perform a thorough diagnosis of the problem through conventional sampling, testing and interpretive techniques. Remedial measures are, therefore, based on opinion tempered with varying degrees of experience. Preventive maintenance in the highway field is an exception rather than the rule. This hit-or-miss, however unavoidable, technique has not been entirely satisfactory. It would appear that improvement of the situation can only be obtained through a time reduction of the evaluation period.

The overall serviceability of a highway system is dependent on a number of factors such as geometrics, traffic volumes and classification, present and future origin-destination requirements, riding qualities, and its structural capacity. A comprehensive highway needs study must include all these factors. This paper considers the evaluation of its structural capacity as an aid in the assessment of maintenance warrants.

Paper sponsored by Committee on Maintenance of Bituminous Pavements and presented at the 45th Annual Meeting.

BASIC CONSIDERATIONS AND TECHNIQUE

Until very recently it has not been possible to categorize the present, let alone predict the probable future structural serviceability of a highway system, on a basis which is sufficiently convenient and rational to be accepted for general use. During the past several years, it has become increasingly apparent that surface deflections obtained under a standard wheel load provide a relative measure of the structural capacity of a highway at a given point and time. Regarded collectively, these measurements can be used to represent its present structural serviceability. Based on experience and performance study correlations, this basically simple data can be employed to portray the present and predict the future structural performance of the entire highway system.

The deflection test is based on the concept that pavement structures which have been conditioned by a given volume and weight of traffic deform elastically under a test load equal to or less than the magnitude of the conditioning traffic. The test is not considered valid when further densification or displacement within the pavement structure or subgrade occurs under the action of the test load.

When a wheel load is applied to the surface of a flexible pavement, the greatest vertical stresses are concentrated near the surface over an area affected by the applied load. These vertical stresses decrease with depth due to a lateral distribution of forces. The incremental layers of soil beneath the wheel load will therefore be compressed according to the intensity of the vertical stress and compressibility of the material. Since vertical stress intensities decrease laterally from the center of the loaded area, the soil within any horizontal layer will likewise be compressed to a lesser degree in a lateral direction. The sum total of the vertical elastic compressions of all incremental layers of soils under the center of a loaded area is represented by the surface deflection, and if such a summation is integrated over the entire surface area affected by the applied load, the product will result in a deflection bowl developed about the loaded area. (See Fig. 1.)

The magnitude of compression within each incremental layer (under a given stress) is a function of the basement soil type and condition, thickness and quality of the surface, base and subbase courses, drainage condition, the relative density, temperature and a number of other minor variables. It will be recognized that these variables are those commonly associated with the structural capacity (strength properties) of a soil. It would, therefore, follow that an integrated measure of the total compression (surface deflection), within a layered system (such as a highway pavement), subjected to a known surface load (wheel load), provides an index of the structural capacity of the system. Due to the large number of variables affecting this deflection, irrespective of how short the highway section may be, variation in deflections from point to point may be expected. Within the assessment of the structural capacity of a highway section, it is therefore necessary to obtain a sufficient number of readings to portray the nature and magnitude of this variation.

Figure 2 shows a typical distribtuion of deflections within a highway section in the form of a histogram showing a reasonable estimate of a normal curve. Consequently, the statistical properties of the normal distribution may be employed within this study.

Since pavement design is a limit design—i. e., there is little or no factor of safety against overstraining of the asphaltic surface—it is necessary to select a control level of deflection for design and load restriction purposes, which will insure that the greatest majority of the given highway section will meet the requirements of projected traffic weights and volumes, and accept a small percentage as underdesigned. This small area of underdesigned pavement will not be critical as it would still be relatively strong and failure would develop very slowly, probably as small areas of depression or roughness which could be readily repaired before serious breaks occurred, and thus the facility can be economically maintained at an acceptable performance level.

The performance of a section of pavement is associated with the weaker areas of that section. Therefore, the deflection level which is exceeded by about 5 or 10 percent of the length of pavement, rather than the average, is more closely associated with the structural performance of the section. Although it is impossible to determine the true

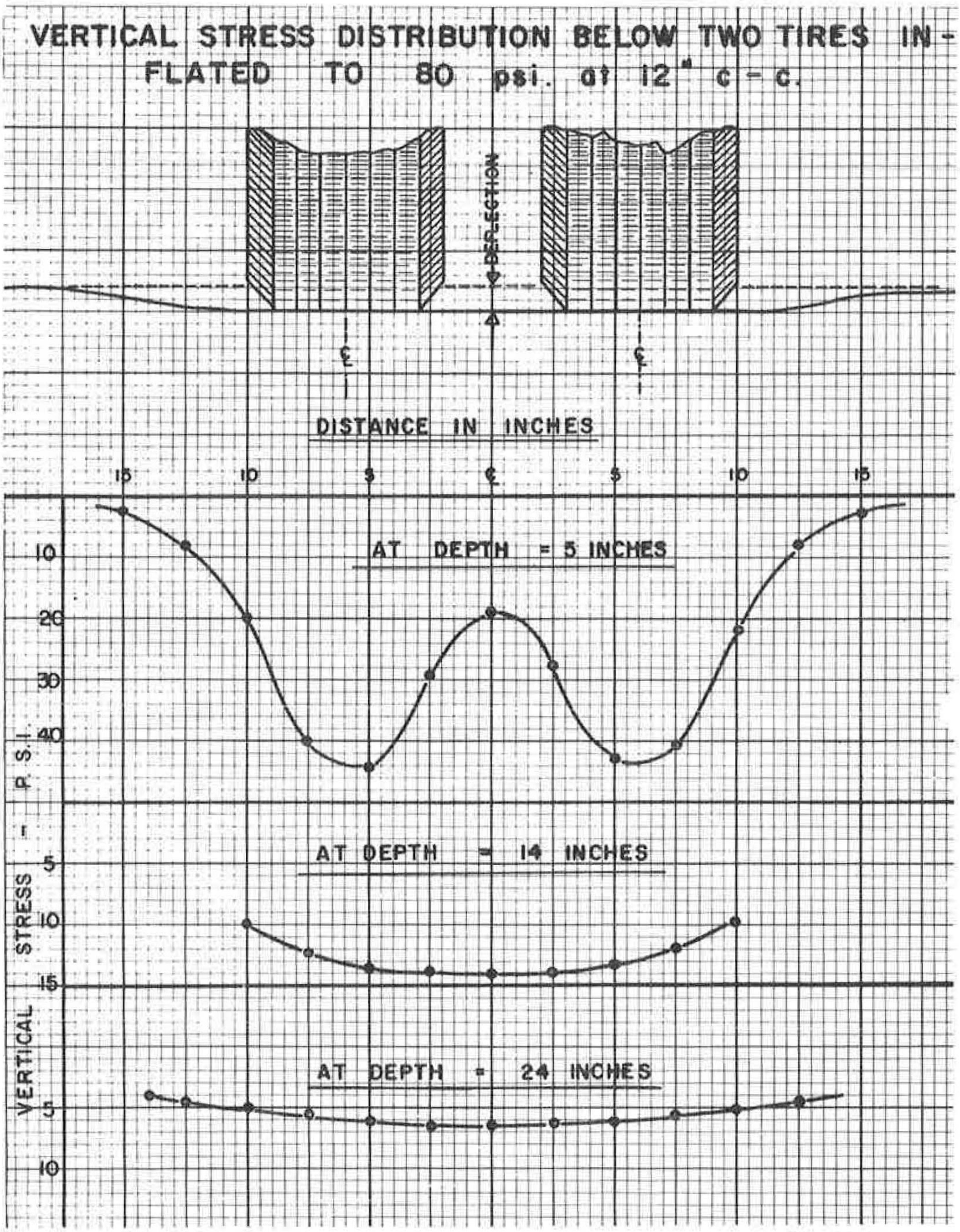
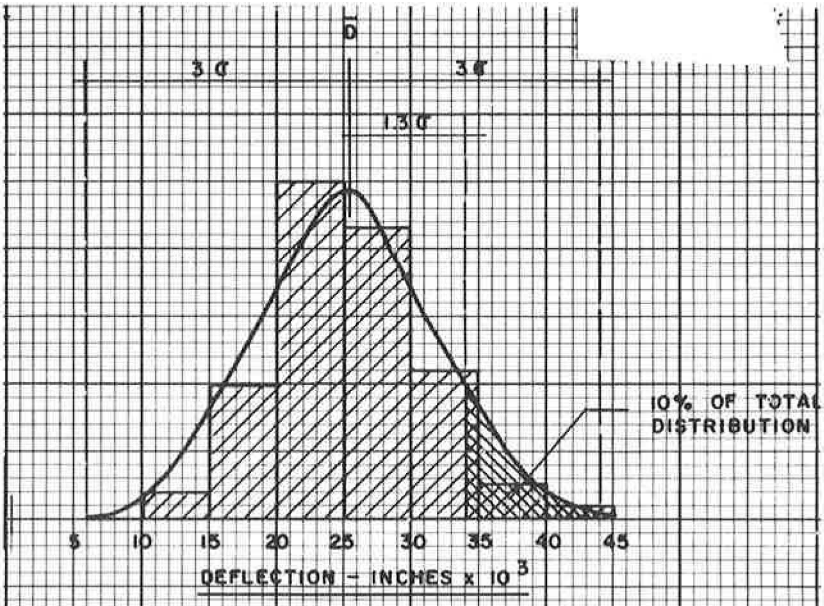


Figure 1.

population mean and standard deviation deflection of a pavement section, we are assured that these values, calculated from readings obtained at 10 or more random points, are good estimates of the true values. The mean and standard deviation thus obtained can be used to determine deflection values corresponding to various probability levels such as shown in the table of Figure 2.



FREQUENCY DISTRIBUTION OF DEFLECTION VALUES FOR A HIGHWAY SECTION WITH AN AVERAGE DEFLECTION, $\bar{D}$ = .0255 INCHES, AND A STANDARD DEVIATION OF, σ = .0065".

DEFLECTION	NORMAL CURVE PARAMETERS APPLIED	PROBABLE PERCENTAGE OF TEST VALUES EXCEEDING GIVEN DEFL.
.0255	$\bar{D}$	50 %
.034	$\bar{D}$ + 1.3 σ	10 %
.036	$\bar{D}$ + 1.6 σ	5 %
.0385	$\bar{D}$ + 2 σ	2 %
.045	$\bar{D}$ + 3 σ	0.1 %

ESTIMATED PROBABLE PERCENTAGE OF ALL POSSIBLE TEST VALUES WITHIN PAVEMENT SECTION EXCEEDING GIVEN DEFLECTIONS.

Figure 2.

For purposes of this study, the 90-percentile level of deflection was selected as representative of the pavement strength limiting the structural capacity of the highway section. Theoretically, the selection of this control level implies that one will allow 10 percent of the section to be overstressed to some degree during its service life and that the authority is prepared to maintain that portion of the 10 percent which may fail from time to time during the life of the pavement.

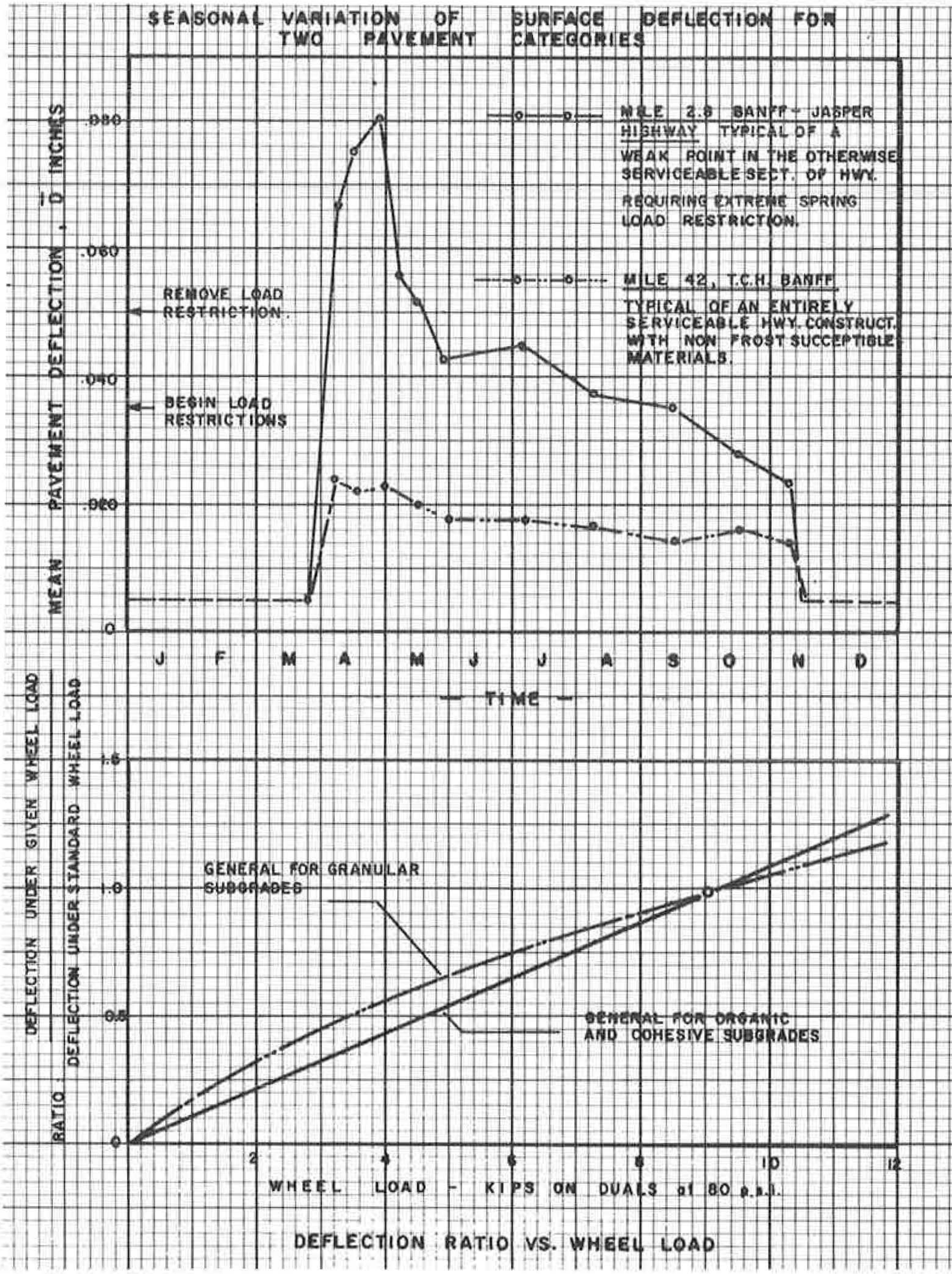


Figure 3.

SEASONAL VARIATION OF PAVEMENT STRENGTH

Under a given load, the maximum deflection of a particular pavement is usually evident within the spring thaw period. The deflection will then decrease to a minimum value by August or September. It has also been observed that a secondary increase in deflection may occur during the latter part of the initial thawing period. This is tentatively attributed to a readjustment in the thermal regime beneath the pavement when some frozen strata still exists at depth. The shape of the deflection curve is dependent on the subgrade soil type, the total thickness and type of pavement structure and the rate and characteristics of the spring thaw (Fig. 3). The cause of this variation is suggested to be the variable amount of water enclosed in the pavement-basement soil system. Upon freezing in late fall and winter, and when accompanied by certain natural conditions of the groundwater supply, sufficient soil pore space and moving frost line, additional water will migrate upwards toward the frozen boundary and freeze there. As thawing progresses, the additional water accumulated during the freezing process is released but may be entrapped by a frozen boundary beneath it. The presence of areas or layers of high water content causes a significant reduction of effective strength within the soil subjected to superimposed wheel loads and results in greater deformation of the structure in order to mobilize equilibrium strength. Thus during the thawing period, we have an accumulation of water in the upper portion of the subgrade and possibly subbase, which results in a decrease in the supporting power of the soil. The relatively rapid release of water during thawing, which depends on the rate of penetration of the thawing isotherm, explains the rapid decrease in bearing strength. The subsequent strength regain is a slower process since the excess water accumulated must be removed in the liquid phase by gravitational drainage or in the vapor phase by evaporation. These processes depend on the excess moisture gradients present and on the permeability of the surrounding soil mass and pavement surface material, respectively.

Tests have shown that for a particular pavement structure a relationship exists between wheel load and deflection. The deflection increases as the wheel load increases on a curvelinear relationship. The load-deflection relationship will vary with the rate of loading, subgrade soil type, the total thickness of pavement structure and the intrinsic properties of the pavement structure, such as type of material, gradation, relative density, etc. This load-deformation relationship, especially for flexible pavements, cannot be defined as a single constant since it depends not only on the fundamental properties of each layer in the system itself, but also on environmental conditions; consequently, these fundamental properties themselves change with season and from year to year.

The CGRA method of deflection testing employed in this survey using a limited period of static loading followed by relatively rapid removal of the load results in a characteristic load-deformation relationship intermediate between the dynamic WASHO method and time-honored plate bearing test. Extensive field testing has shown that the CGRA method is fast and accurate and that results are reproducible on extremely soft pavement and weak subgrade conditions. This is important during spring testing when a large loss in strength is experienced in some subgrades. It is imperative that the method of test is consistent for all sections if spring restrictions are being set on the basis of these tests. The measured strength of the subgrade soil is dependent on the rate of strain applied during loading. This rate of strain is particularly important in tests during the spring when the subgrade is in its weakest condition. The CGRA static method of test minimizes the effect of this variable.

Use of deflection criteria in setting spring load restrictions usually results in restrictions being set slightly later and lifted later than with the old technique of setting restrictions by opinion. Deflection experience to date indicates that setting load restrictions by opinion usually results in restrictions being set too early and lifted at a time when pavement deflection was most critical. Deflection criteria enable load restrictions to be set to protect the pavement structure and also permit the correct percentage of restriction to be imposed. For example, many highways which would have had a 50 percent restriction, may now have a 75 percent restriction (75 percent of normal allowable load) or may require no restrictions.

Deflection data on a highway route give an accurate picture of its strength characteristics at the time of testing. A particular highway may contain only one or two weak sections of limited length. With deflection data it is possible to assess the amount of maintenance required on the weak sections to avoid a lengthy ban. Strengthening of these weak areas may eliminate all ban requirements.

CRITICAL DEFLECTION VALUES

When we speak of strength, we mean the ability of the pavement structure to sustain superimposed loadings without permanent deformation. Excessive deformation may result in rutting, shoving or any one of a variety of surface cracking patterns. Every pavement system will exhibit a yield point or critical value of internal strain beyond which permanent deformation or rupture will occur.

Obviously, critical deflection values must be set at a level less than that at which permanent deformation or rupture of any element of the pavement will occur.

The properties of the surface are dependent on temperature, and the supporting capacity of the subgrade depends on the content and distribution of moisture which varies seasonally and throughout the life of the pavement. Thus the critical deflection to be selected for any single pavement type is a time-dependent variable. This latter factor is not too important in concrete surfaces since the material itself has the ability to distribute load over a larger area when subsurface support is reduced. The bituminous pavement derives its ability to sustain loads directly from the base and subgrade and its capacity to deform without fracture depends on the temperature. Since the effect of lower temperature is to render the relatively thin bituminous surfacing more brittle, the pavement is more susceptible to detrimental cracking during the spring thawing period when the surface layer cannot deform as readily without rupture. It would be impractical to define a critical deflection value for each pavement system. A field analysis must therefore be directed toward determining an average, safe value of certain generalized groups of pavement structures. Analysis of the WASHO Road Test data suggested critical deflection values of 0.035 and up to 0.050 in. for spring and summer conditions, respectively. These are average values based on the observed distress of several different pavement designs placed on one type of subgrade. Higher deflection values are usually permissible on a lightly traveled highway as compared to a heavily traveled highway for the same magnitude of wheel loads. Some pavements are still in service today with deflections as high as 0.075 in. but carrying relatively low-volume traffic. However, in view of the difficulty of defining an exact critical deflection value for each pavement type, the critical values established at the WASHO test road are considered most practical for present use.

GENERAL DISCUSSION ON PROCEDURE

As is the case with most problems of this nature, it is desirable to obtain an inventory of the existing highway system. For this detailed inventory the highway routes are divided into uniform sections, ranging from ½ to 5 miles in length, following the establishment of the boundaries of the following variables:

1. Maintenance districts,
2. Construction history limits,
3. Subgrade soil type (natural),
4. Imported subgrade soil type,
5. Drainage conditions,
6. Age of original surfacing,
7. Age of resurfacing,
8. Heavy axle coverage per lane,
9. Current A.A.D.T.,
10. Pavement lane width,
11. Bituminous surface thickness and type,
12. Base course thickness,
13. Subbase thickness,
14. Shoulder type and width,
15. Rainfall,
16. Freezing index, and
17. Height of grade above natural ground.

The mean and standard deviation of the "full" deflections and present performance rating are then measured for each section.

The completed inventory is studied to locate critical sections which are representative of the highway route requiring spring load restrictions. These sections should be representative of soil types, traffic conditions, pavement design and type, and freezing indices. The deflections are then taken in these sections to establish the seasonal variation in strength. An 18,000-lb single-axle load is used to determine this time-deflection relationship. The number of sections to be selected on a particular route will vary with the aforementioned conditions but are kept to a minimum. Testing should commence shortly before thawing and continue until the pavement has started to recover strength. Measurements are taken at intervals of four to five days while load restrictions are in force, so that a minimum of delay may be experienced in lifting restrictions. Each time a section is tested, ten new random test points are selected in the outer wheelpaths of the section. The average deflection value is then obtained for the section from the ten test results. To expedite testing, this procedure was varied by locating and marking one set of random points and repeating the periodic tests at these locations. As the mean deflection values begin to stabilize in the late spring or early summer the time between reading is increased to monthly intervals.

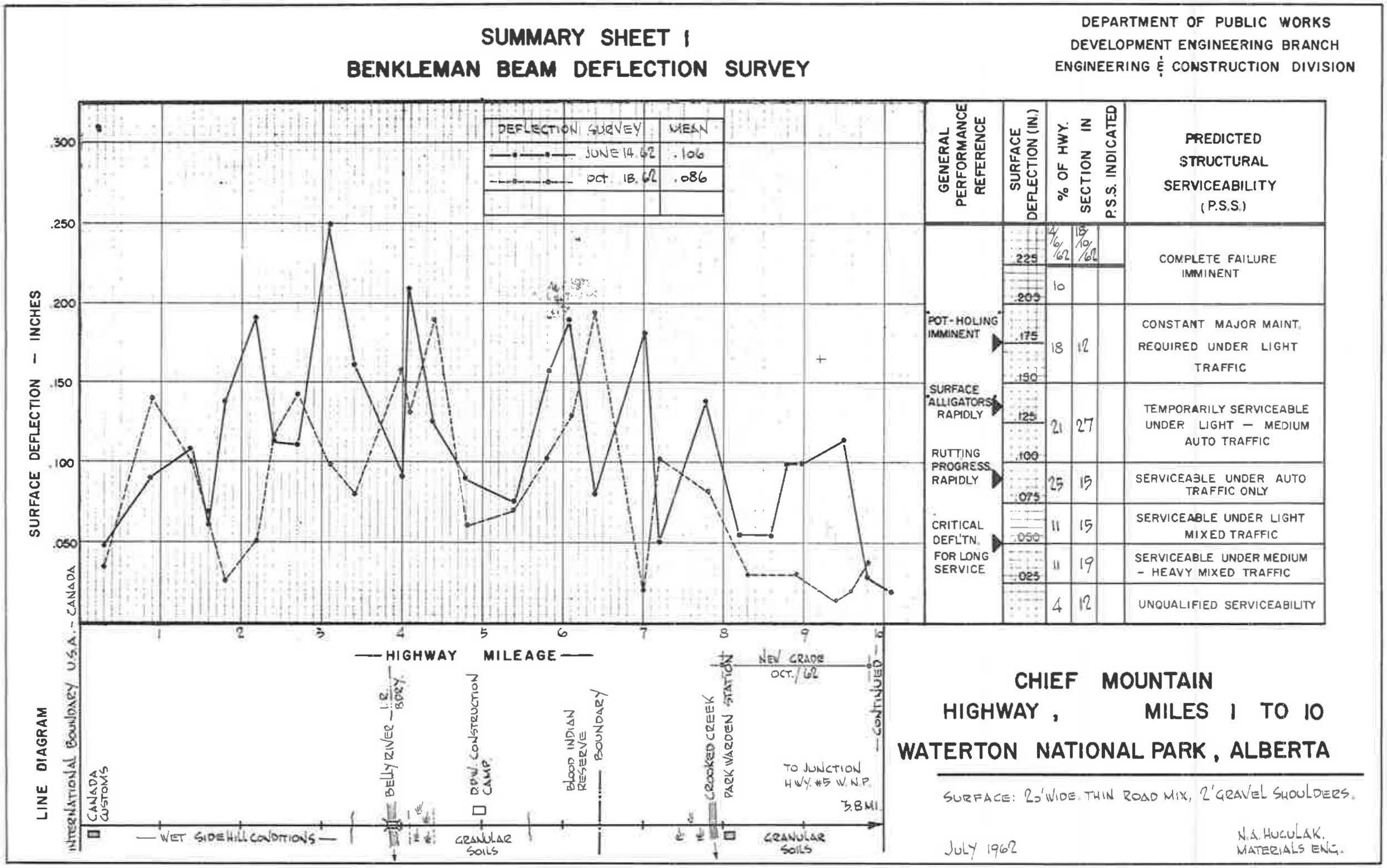
Pavement inventories, as described above, have been obtained by most highway agencies in Canada during Stage I of the work undertaken by the Pavement Design and Evaluation Committee. A majority of the data necessary for this purpose is therefore available and may be extracted from the printouts of Stage I. For other highways, i.e., those not included in the detailed inventory, deflections at three to four random points per mile were obtained in the outer wheelpath. General notes on the performance of the surface at each deflection point should also be taken to assist in the interpretation of these results. One crew can cover about 25 miles per day in obtaining these results. Deflections during the spring breakup period need only be obtained on highway routes which are judged susceptible to frost action and would therefore lose a high percentage of their normal load-carrying capacity.

The results of this survey are plotted in the form of deflection profiles (Figs. 3 through 7), usually in 10-mi sections. Out of several hundred miles covered within the first year of this work, five sections covering a wide range of highway categories (in terms of their structural serviceability) are included in this paper. The summary sheets (Figs. 4-8) include a graphic-tabular interpretation of these readings for ready assessment of the general structural serviceability of the section. The spectrum of readings is divided into various levels of deflection and their percentage noted opposite the appropriate structural serviceability (traffic handling ability) category. The column entitled "General Performance Reference" is applicable for highways carrying medium-heavy mixed traffic and should only be used in conjunction with the surface deflection scale and not with the last column entitled "Structural Serviceability."

A considerable amount of information concerning each highway section can be obtained by close examination of the summary sheet. For example, if we refer to Summary Sheet #1 (Fig. 4), covering miles 1 to 10 of the Chief Mountain Highway, we find the following:

1. From the table: (a) in spring complete failure is imminent for 10 percent of the section; (b) constant major maintenance is required under light traffic over 18 percent of the section in spring and 12 percent of the section under fall conditions; (c) 27 and 21 percent of the section are temporarily serviceable under light-medium auto traffic under fall and spring conditions, respectively; (d) 15 and 25 percent of the section are serviceable under auto traffic under fall and spring conditions, respectively; (e) 15 and 11 percent of the section are serviceable under light mixed traffic under fall and spring conditions, respectively; (f) 19 and 11 percent of the section are serviceable under medium-heavy mixed traffic under spring and fall conditions, respectively; (g) 12 and 4 percent of the section is serviceable without reservation under all conditions of traffic under fall and spring conditions, respectively; and (h) the critical deflection of 0.050 in. is exceeded in approximately 80 percent of the section under truck traffic.

2. During the spring the mean deflection is equal to 0.106 in. with a standard deviation of 0.052 in., and during the fall period the mean deflection is equal to 0.086 in. with a standard deviation of 0.041 in.


SUMMARY SHEET 1
BENKLEMAN BEAM DEFLECTION SURVEY
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
ENGINEERING & CONSTRUCTION DIVISION
DEFLECTION SURVEY
MEAN
JUNE 14. 62
.106
OCT 18. 62
.086
SURFACE DEFLECTION - INCHES
.300
.250
.200
.150
.100
.050
GENERAL PERFORMANCE REFERENCE
SURFACE DEFLECTION (IN.)
% OF HWY. SECTION IN P.S.S. INDICATED
PREDICTED STRUCTURAL SERVICEABILITY (P.S.S.)
POT-HOLING IMMINENT
SURFACE ALLIGATORS RAPIDLY
RUTTING PROGRESS RAPIDLY
CRITICAL DEFL'TN. FOR LONG SERVICE
COMPLETE FAILURE IMMINENT
CONSTANT MAJOR MAINT. REQUIRED UNDER LIGHT TRAFFIC
TEMPORARILY SERVICEABLE UNDER LIGHT - MEDIUM AUTO TRAFFIC
SERVICEABLE UNDER AUTO TRAFFIC ONLY
SERVICEABLE UNDER LIGHT MIXED TRAFFIC
SERVICEABLE UNDER MEDIUM - HEAVY MIXED TRAFFIC
UNQUALIFIED SERVICEABILITY
HIGHWAY MILEAGE
LINE DIAGRAM
INTERNATIONAL BOUNDARY U.S.A. - CANADA
CANADA CUSTOMS
WET SIDEHILL CONDITIONS
BELLY RIVER - I.R. BDRY.
D.P.W. CONSTRUCTION CAMP
GRANULAR SOILS
BLOOD INDIAN RESERVE BOUNDARY
CROOKED CREEK
PARK WARDEN STATION
NEW GRADE OCT./62
CONTINUED
TO JUNCTION HWY. #5 W.N.P.
3.8 MI.
CHIEF MOUNTAIN HIGHWAY, MILES 1 TO 10
WATERTON NATIONAL PARK, ALBERTA
SURFACE: 22' WIDE THIN ROAD MIX, 2' GRAVEL SHOULDERS.
JULY 1962
N.A. HUCULAK.
MATERIALS ENG.

F' 4.

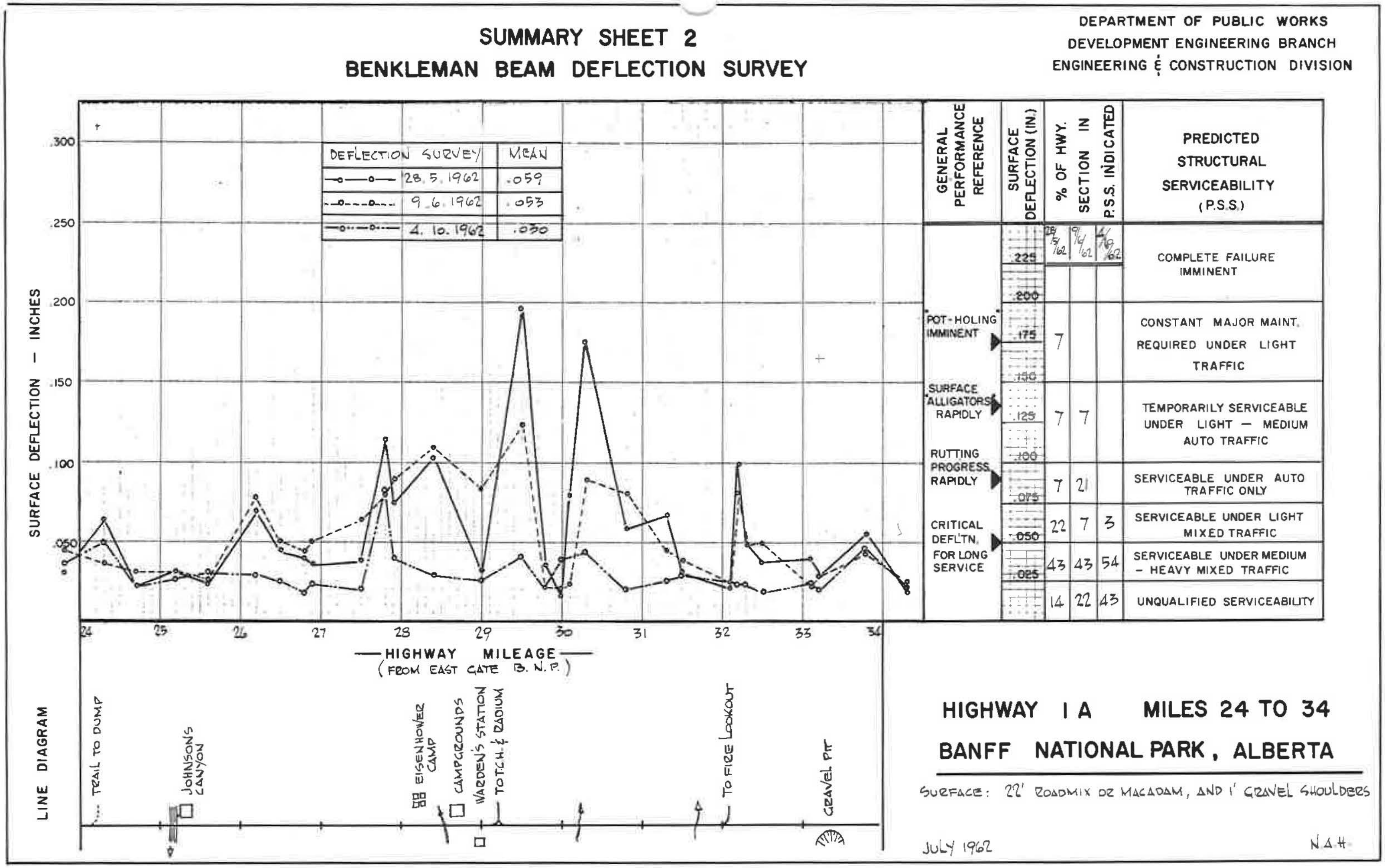

SUMMARY SHEET 2
BENKLEMAN BEAM DEFLECTION SURVEY
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
ENGINEERING & CONSTRUCTION DIVISION
DEFLECTION SURVEY | MEAN
28.5.1962 | .059
9.6.1962 | .053
4.10.1962 | .030
SURFACE DEFLECTION — INCHES
.300 .250 .200 .150 .100 .050
24 25 26 27 28 29 30 31 32 33 34
HIGHWAY MILEAGE
(FROM EAST GATE B.N.P.)
GENERAL PERFORMANCE REFERENCE
SURFACE DEFLECTION (IN.)
% OF HWY. SECTION IN P.S.S. INDICATED
PREDICTED STRUCTURAL SERVICEABILITY (P.S.S.)
28/5/62 9/6/62 4/10/62
.225 .200 .175 .150 .125 .100 .075 .050 .025
COMPLETE FAILURE IMMINENT
POT-HOLING IMMINENT
7
CONSTANT MAJOR MAINT. REQUIRED UNDER LIGHT TRAFFIC
SURFACE ALLIGATORS RAPIDLY
7 7
TEMPORARILY SERVICEABLE UNDER LIGHT — MEDIUM AUTO TRAFFIC
RUTTING PROGRESS RAPIDLY
7 21
SERVICEABLE UNDER AUTO TRAFFIC ONLY
CRITICAL DEFL'TN. FOR LONG SERVICE
22 7 3
SERVICEABLE UNDER LIGHT MIXED TRAFFIC
43 43 54
SERVICEABLE UNDER MEDIUM - HEAVY MIXED TRAFFIC
14 22 43
UNQUALIFIED SERVICEABILITY
LINE DIAGRAM
TRAIL TO DUMP
JOHNSONS CANYON
EISENHOWER CAMP
CAMPGROUNDS
WARDEN'S STATION
TOTCH. & RADIUM
TO FIRE LOOKOUT
GRAVEL PIT
HIGHWAY 1 A MILES 24 TO 34
BANFF NATIONAL PARK, ALBERTA
SURFACE: 22' ROADMIX OR MACADAM, AND 1' GRAVEL SHOULDERS
JULY 1962
N.A.H.

Figure 5.

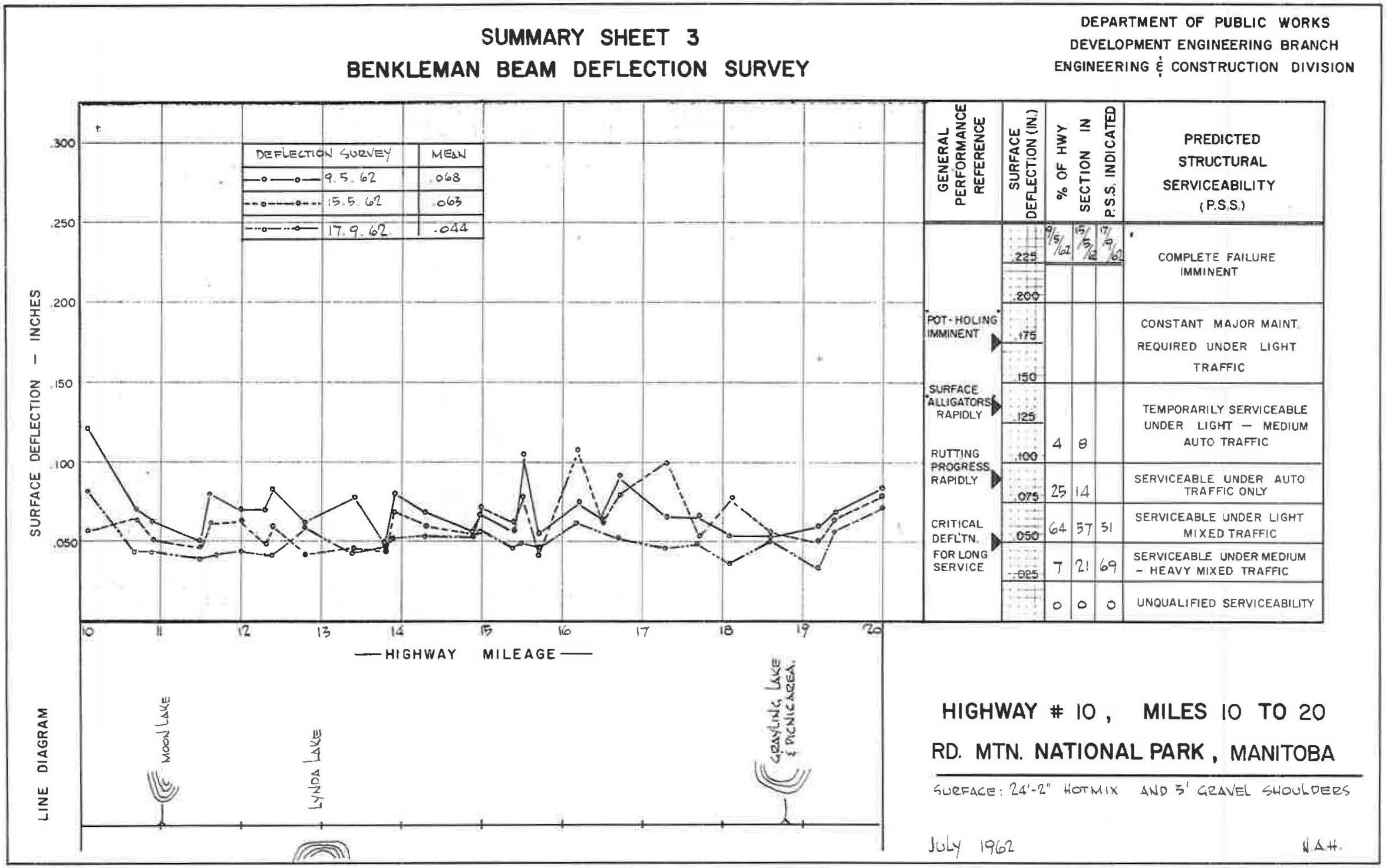

SUMMARY SHEET 3
BENKLEMAN BEAM DEFLECTION SURVEY
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
ENGINEERING & CONSTRUCTION DIVISION
DEFLECTION SURVEY
MEAN
9.5.62
.068
15.5.62
.063
17.9.62
.044
SURFACE DEFLECTION - INCHES
.300
.250
.200
.150
.100
.050
GENERAL PERFORMANCE REFERENCE
SURFACE DEFLECTION (IN.)
% OF HWY SECTION IN P.S.S. INDICATED
PREDICTED STRUCTURAL SERVICEABILITY (P.S.S.)
9/5/62
15/5/62
17/9/62
.225
.200
COMPLETE FAILURE IMMINENT
POT-HOLING IMMINENT
.175
.150
CONSTANT MAJOR MAINT. REQUIRED UNDER LIGHT TRAFFIC
SURFACE ALLIGATORS RAPIDLY
.125
.100
4 8
TEMPORARILY SERVICEABLE UNDER LIGHT — MEDIUM AUTO TRAFFIC
RUTTING PROGRESS RAPIDLY
.075
25 14
SERVICEABLE UNDER AUTO TRAFFIC ONLY
CRITICAL DEFL'TN. FOR LONG SERVICE
.050
64 57 31
SERVICEABLE UNDER LIGHT MIXED TRAFFIC
.025
7 21 69
SERVICEABLE UNDER MEDIUM - HEAVY MIXED TRAFFIC
0 0 0
UNQUALIFIED SERVICEABILITY
10 11 12 13 14 15 16 17 18 19 20
HIGHWAY MILEAGE
LINE DIAGRAM
MOON LAKE
LYNDA LAKE
GRAYLING LAKE & PICNIC AREA.
HIGHWAY # 10, MILES 10 TO 20
RD. MTN. NATIONAL PARK, MANITOBA
SURFACE: 24'-2" HOTMIX AND 3' GRAVEL SHOULDERS
JULY 1962
W.A.H.

Figure 6.

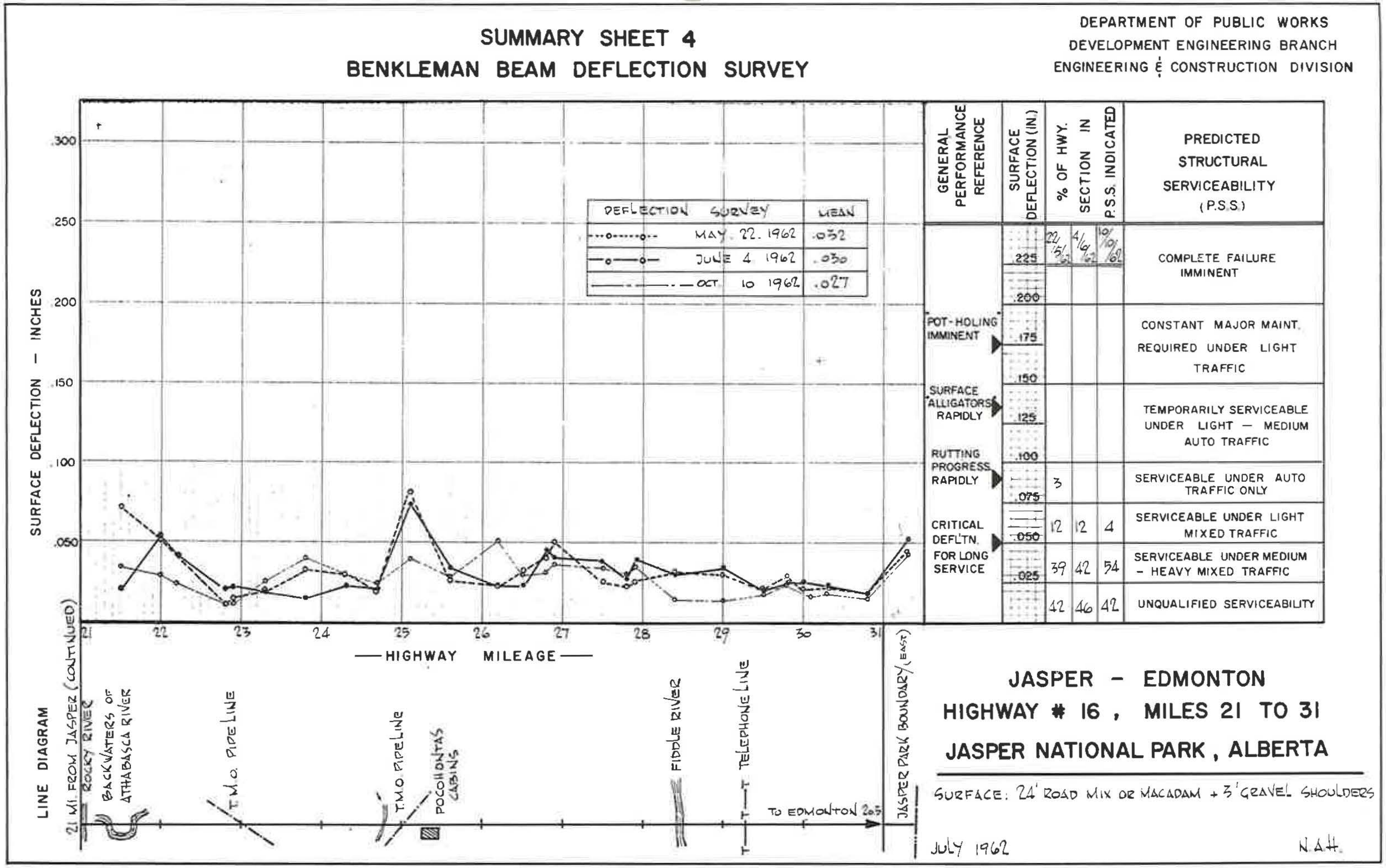
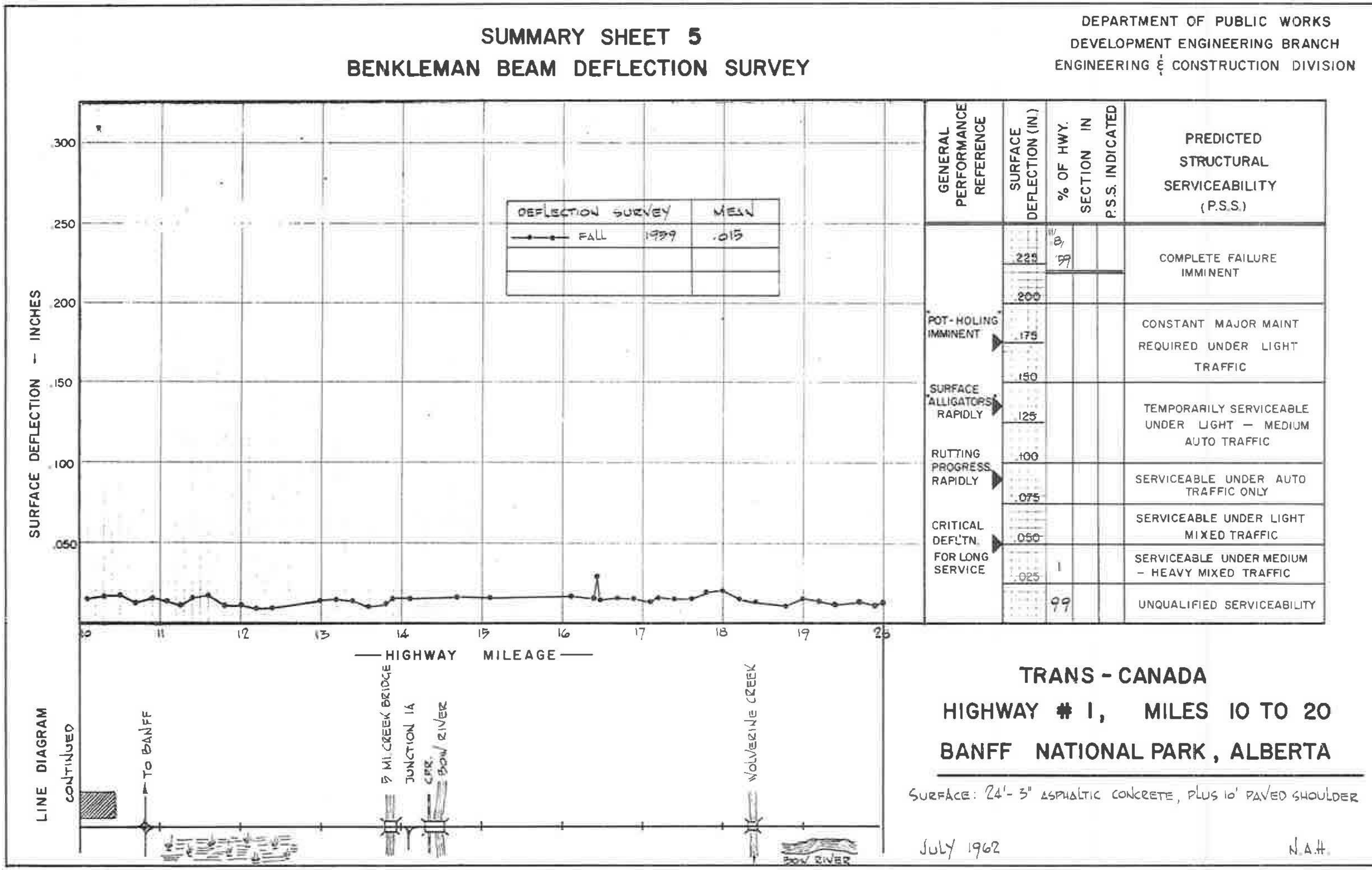

SUMMARY SHEET 4
BENKLEMAN BEAM DEFLECTION SURVEY
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
ENGINEERING & CONSTRUCTION DIVISION
SURFACE DEFLECTION - INCHES
.300
.250
.200
.150
.100
.050
DEFLECTION SURVEY
MEAN
MAY 22. 1962 .032
JUNE 4 1962 .030
OCT. 10 1962 .027
21
22
23
24
25
26
27
28
29
30
31
HIGHWAY MILEAGE
GENERAL PERFORMANCE REFERENCE
SURFACE DEFLECTION (IN.)
% OF HWY. SECTION IN P.S.S. INDICATED
PREDICTED STRUCTURAL SERVICEABILITY (P.S.S.)
22/5/62
4/6/62
10/10/62
.225
COMPLETE FAILURE IMMINENT
.200
POT-HOLING IMMINENT
.175
CONSTANT MAJOR MAINT. REQUIRED UNDER LIGHT TRAFFIC
.150
SURFACE ALLIGATORS RAPIDLY
.125
TEMPORARILY SERVICEABLE UNDER LIGHT - MEDIUM AUTO TRAFFIC
RUTTING PROGRESS RAPIDLY
.100
3
SERVICEABLE UNDER AUTO TRAFFIC ONLY
.075
CRITICAL DEFL'TN. FOR LONG SERVICE
12 12 4
SERVICEABLE UNDER LIGHT MIXED TRAFFIC
.050
39 42 54
SERVICEABLE UNDER MEDIUM - HEAVY MIXED TRAFFIC
.025
42 46 42
UNQUALIFIED SERVICEABILITY
LINE DIAGRAM
21 MI. FROM JASPER (CONTINUED)
ROCKY RIVER
BACKWATERS OF ATHABASCA RIVER
T.M.O. PIPE LINE
T.M.O. PIPELINE
POCOHONTAS CABINS
FIDDLE RIVER
TELEPHONE LINE
TO EDMONTON 205
JASPER PARK BOUNDARY (EAST)
JASPER - EDMONTON
HIGHWAY # 16, MILES 21 TO 31
JASPER NATIONAL PARK, ALBERTA
SURFACE: 24' ROAD MIX OR MACADAM + 3' GRAVEL SHOULDERS
JULY 1962
N.A.H.

Figure 7.


SUMMARY SHEET 5
BENKLEMAN BEAM DEFLECTION SURVEY
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
ENGINEERING & CONSTRUCTION DIVISION
SURFACE DEFLECTION - INCHES
.300
.250
.200
.150
.100
.050
DEFLECTION SURVEY
MEAN
FALL 1959
.015
10
11
12
13
14
15
16
17
18
19
20
HIGHWAY MILEAGE
GENERAL PERFORMANCE REFERENCE
SURFACE DEFLECTION (IN.)
% OF HWY. SECTION IN P.S.S. INDICATED
PREDICTED STRUCTURAL SERVICEABILITY (P.S.S.)
POT-HOLING IMMINENT
SURFACE ALLIGATORS RAPIDLY
RUTTING PROGRESS RAPIDLY
CRITICAL DEFL'TN. FOR LONG SERVICE
.225
.200
.175
.150
.125
.100
.075
.050
.025
1
99
COMPLETE FAILURE IMMINENT
CONSTANT MAJOR MAINT REQUIRED UNDER LIGHT TRAFFIC
TEMPORARILY SERVICEABLE UNDER LIGHT - MEDIUM AUTO TRAFFIC
SERVICEABLE UNDER AUTO TRAFFIC ONLY
SERVICEABLE UNDER LIGHT MIXED TRAFFIC
SERVICEABLE UNDER MEDIUM - HEAVY MIXED TRAFFIC
UNQUALIFIED SERVICEABILITY
LINE DIAGRAM
CONTINUED
TO BANFF
5 MI. CREEK BRIDGE
JUNCTION 1A
CRK.
BOW RIVER
WOLVERINE CREEK
BOW RIVER
TRANS - CANADA
HIGHWAY # 1, MILES 10 TO 20
BANFF NATIONAL PARK, ALBERTA
SURFACE: 24'- 3" ASPHALTIC CONCRETE, PLUS 10' PAVED SHOULDER
JULY 1962
N.A.H.

F' ·re 8.

3. Besides giving the observer a quick general impression of the structural capacity of the highway, the deflection profile indicates that there are numerous weak areas within the section and where these weak areas are located.

4. Assuming that this section was required to carry light mixed traffic (say less than 300 cars, 15 buses and 5 heavy trucks daily), it can readily be seen that over 50 percent of the highway is not capable of performing under these conditions over a reasonable period of time. Thus one would conclude that severe load restrictions are required or that reconstruction of the section must be carried out in the very near future.

As mentioned previously, a study of the deflection profile will readily show the areas of greatest weakness. Where deflections much higher than the average occur over short isolated stretches such as miles 29-31 of Summary Sheet #2 (Fig. 5), consideration should be given to strengthen these points to a level at least equivalent to adjacent areas. This would reduce spring load reduction requirements significantly and possibly eliminate a reoccurring problem area.

Summary Sheet #3 (Fig. 6) is a good example of a highway section which is "uniformly weak." This section is quite capable of handling light mixed traffic except during the breakup season during which time severe load restrictions are warranted. It is typical of a substandard pavement whose structural serviceability can be raised with subexcavation and backfill with nonfrost-susceptible materials at a small number of short areas and the application of an overlay of several inches of gravel and a thin surface course.

The type of highway summarized on sheet #4 (Fig. 7) is very similar to that described for sheet #3 except that weak areas are more isolated and overlay load restriction requirements are not as great.

Summary Sheet #5 (Fig. 8) is an example of a highway section which may be described as serviceable without reservation, and which will perform for a long period with little maintenance under most traffic conditions.

As stated earlier, it is apparent that surface deflections obtained under a standard wheel load provide a relative measure of the structural capacity of a highway at a given point and time. Regarded collectively, these measurements can be used to represent its present structural serviceability. Based on experience and performance study correlations, these basically simple data can be employed to portray the present and probable future structural performance and maintenance requirements of the entire highway system.

A review of the deflection profiles by one who is familiar with the performance of the various highways will show that the average deflections alone do not provide an adequate index of structural capacity. This is not unexpected since the mean value only tells us that 50 percent of the data were below and 50 percent of the data were above that level of deflection. Obviously, two sections of highway which have the same average deflection but very different standard deviations of deflection will perform radically different under a given set of traffic conditions. The highway section with a high standard deviation will have many soft areas which will fail early. A more accurate representation of the structural serviceability of the section is therefore obtained when some significance is given to the degree by which the various deflections along the route vary about the mean value (standard deviation).

A Highway Category and Action Chart (Fig. 9) has been designed as a further interpretation and application of the data obtained in this survey. The chart is entered at the mean and standard deviation values of deflection for each section of highway. The other properties and categories of the section are then picked off the appropriate scales.

For example, if we refer to Summary Sheet #3 covering highway 10, miles 10-20, we find that this section has a mean fall deflection value of 0. 044 in. with a standard deviation of 0. 014 in. Upon entering the chart at these values we find that this section of highway: (a) requires a spring load reduction of 50 percent; (b) is capable of carrying an axle load of 9. 0 kips without distress during the spring; (c) has a 90 percentile deflection level of 0. 060 in. during the fall period; (d) requires the addition of eight "inches of gravel" plus two inches of asphaltic surface if spring reductions are to be

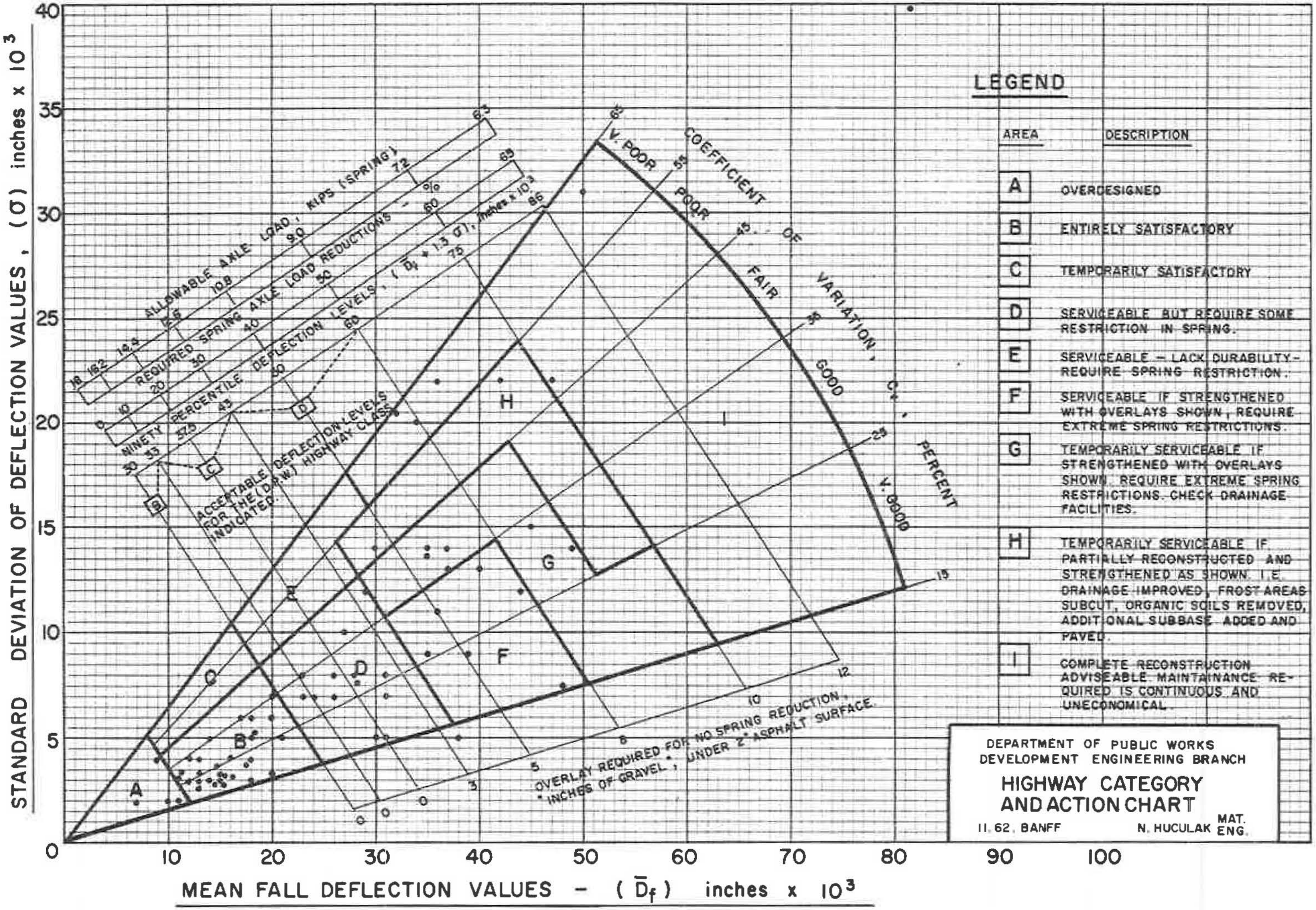

STANDARD DEVIATION OF DEFLECTION VALUES, (σ) inches x 10³
MEAN FALL DEFLECTION VALUES – ($\bar{D}_f$) inches x 10³
0
5
10
15
20
25
30
35
40
10
20
30
40
50
60
70
80
90
100
ALLOWABLE AXLE LOAD, KIPS (SPRING)
REQUIRED SPRING AXLE LOAD REDUCTIONS – %
NINETY PERCENTILE DEFLECTION LEVELS, ($\bar{D}_f$ + 1.3 σ), inches x 10³
ACCEPTABLE DEFLECTION LEVELS FOR THE (D.P.W.) HIGHWAY CLASS INDICATED.
COEFFICIENT OF VARIATION, Cv, PERCENT
V. POOR
POOR
FAIR
GOOD
V. GOOD
OVERLAY REQUIRED FOR NO SPRING REDUCTION, INCHES OF GRAVEL, UNDER 2" ASPHALT SURFACE.
A
B
C
D
E
F
G
H
I
LEGEND
AREA
DESCRIPTION
A OVERDESIGNED
B ENTIRELY SATISFACTORY
C TEMPORARILY SATISFACTORY
D SERVICEABLE BUT REQUIRE SOME RESTRICTION IN SPRING.
E SERVICEABLE – LACK DURABILITY – REQUIRE SPRING RESTRICTION.
F SERVICEABLE IF STRENGTHENED WITH OVERLAYS SHOWN, REQUIRE EXTREME SPRING RESTRICTIONS.
G TEMPORARILY SERVICEABLE IF STRENGTHENED WITH OVERLAYS SHOWN. REQUIRE EXTREME SPRING RESTRICTIONS. CHECK DRAINAGE FACILITIES.
H TEMPORARILY SERVICEABLE IF PARTIALLY RECONSTRUCTED AND STRENGTHENED AS SHOWN. I.E. DRAINAGE IMPROVED, FROST AREAS SUBCUT, ORGANIC SOILS REMOVED, ADDITIONAL SUBBASE ADDED AND PAVED.
I COMPLETE RECONSTRUCTION ADVISEABLE. MAINTAINANCE REQUIRED IS CONTINUOUS AND UNECONOMICAL.
DEPARTMENT OF PUBLIC WORKS
DEVELOPMENT ENGINEERING BRANCH
HIGHWAY CATEGORY AND ACTION CHART
11. 62. BANFF
N. HUCULAK MAT. ENG.

Figure 9.

avoided; (e) has a structural coefficient of variation, C_V = 33 percent, which is in the "good" category—implying that the quality control during construction was adequate and that areas of extreme weakness are probably limited; and (f) is temporarily serviceable if partially reconstructed and strengthened as shown.

The position of more than 60 sections each representing 10 miles of highway is shown on the chart.

The chart is considered more applicable to highways carrying medium-mixed traffic, although recognition is given to the acceptance of higher deflections for the lower highway classes.

In general, the chart is a comprehensive summary of recent developments in the field of pavement design and evaluation. The location of the boundaries identifying the nine categories (areas on the chart) is an expression of experience with design, construction and maintenance of several hundred miles of highway ranging from minor access roads to the primary thoroughfare.

Every highway system will always contain sections which will fall within most categories shown on the chart. A large percentage will hopefully fall within the category described as "entirely satisfactory" (such as area "B") with 90 percentile deflection levels between 0.015 and 0.030 and coefficient of variation between 15 and 45 percent. These highways will obviously require very little or no structural maintenance over a considerable period of service. At the other end of the scale another percentage of the system will be obvious candidates for reconstruction and will fall in area "I," with 90 percentile deflection levels greater than 0.075 in. The maintenance authority is therefore less concerned with these routes.

The highway categories with which a maintenance authority will be mostly concerned will vary from department to department since this will depend on economics and policy to some extent. It is expected, however, that in the majority they will be those which fall in areas "D" to "H" inclusive, i.e., with 90 percentile deflection levels between 0.030 and 0.075 in. Obviously sections which fall in area "D" will require less attention than those which are located in "F" through to "H" as briefly described on the chart.

During development of this chart, an attempt was made to establish a "Useful Life Scale" opposite the 90 percentile fall deflection levels (0.030 to 0.090 in.). This would be very useful in highway planning since one could then predict when each section may have to be programmed for reconstruction or improvement. Since these figures did not correlate too well, the scale was omitted. Generally, however, all highways within area "B" are less than 5 years old, sections within area "D" are 4 to 10 years old, areas "G" and "H," 8 to 15 years old, and area "I," 13 to 20 years old.

Assuming that traffic conditions were reasonably uniform throughout the system, a sequence of reconstruction requirements may be established from the chart by the relative position of the various sections on the diagram. At some time during the fiscal year, maintenance and construction requirements for future work are submitted to the central office from the districts. The administrator endeavoring to establish the program for the entire system may find the chart of some value in this regard.

If similar surveys are carried out on these roads in future years, it will be interesting to note how their position on the chart changes with time. It is expected, however, that the variation in strength from point to point within a section of pavement which is built into it at the time of construction remains constant and therefore a pavement will "age" at a constant coefficient of variation.

Benkelman Beam—Auxiliary Instrument of the Maintenance Engineer

F. BOLIVAR LOBO CARNEIRO
Materials Engineer, Guanabara Highway Department and Consultant Engineer, Proenge Ltd.

•ONE of the most difficult tasks of the maintenance engineer is to anticipate pavement maintenance requirements; it is necessary for him to have an accurate knowledge of the structural performance of each section of the pavement.

The aim of this paper is to demonstrate how deflections measured by a Benkelman beam can assist the maintenance engineer in evaluating the structural condition of pavements in terms of required maintenance.

This paper describes methods of measuring deflection using the Benkelman beam, analyzes the disadvantages of the methods used on the WASHO Road Test, and presents the method used by the Canadian Good Roads Association, which we propose be standardized and adopted in Brazil.

The author also discusses modern methods currently used to design and plan pavement overlays, and describes the types of data which must be obtained in Brazil.

PAVEMENT DEFORMATIONS

Before examining deflection measurements made with the Benkelman beam, it is necessary to describe the main types of deformation to which pavements are subjected.

All pavements, however well-designed and constructed, suffer small deformations caused principally by the action of wheel loads. These deformations can be classified as permanent and transient (Figs. 1 and 2); the latter in turn can be subdivided into elastic and viscoelastic.

Permanent Deformations

The most common permanent deformations of pavement are consolidation and plastic deformation.

Consolidation. —The consolidation of pavement layers or of the subgrade caused by traffic after the pavement's construction produces ruts in the wheelpaths. This type of permanent deformation does not generally cause surface cracks but produces some riding discomfort. The consolidation caused by traffic can be reduced to a minimum by adequately compacting the pavement's layers and subgrade.

Plastic Deformation. —Plastic deformation, also caused by traffic, produces permanent pavement deformations similar to consolidation.

The deformation produced by consolidation causes a reduction in the volume of layers of the pavement or of the subgrade. However, plastic deformation causes only a change of shape, without variation of volume. Whereas consolidation diminishes and tends toward stabilization, plastic flow progresses with the action of the vehicle loads. In plastic deformation the deformed layer decreases in thickness in the areas of the wheelpaths and increases in thickness in the surrounding areas.

Plastic deformation only occurs when one of the layers fails by shearing due to a load in excess of its bearing capacity. Such deformations occur at the edges of underdesigned pavements or on pavements constructed with inadequate material.

Paper sponsored by Committee on Maintenance of Bituminous Pavements and presented at the 45th Annual Meeting.

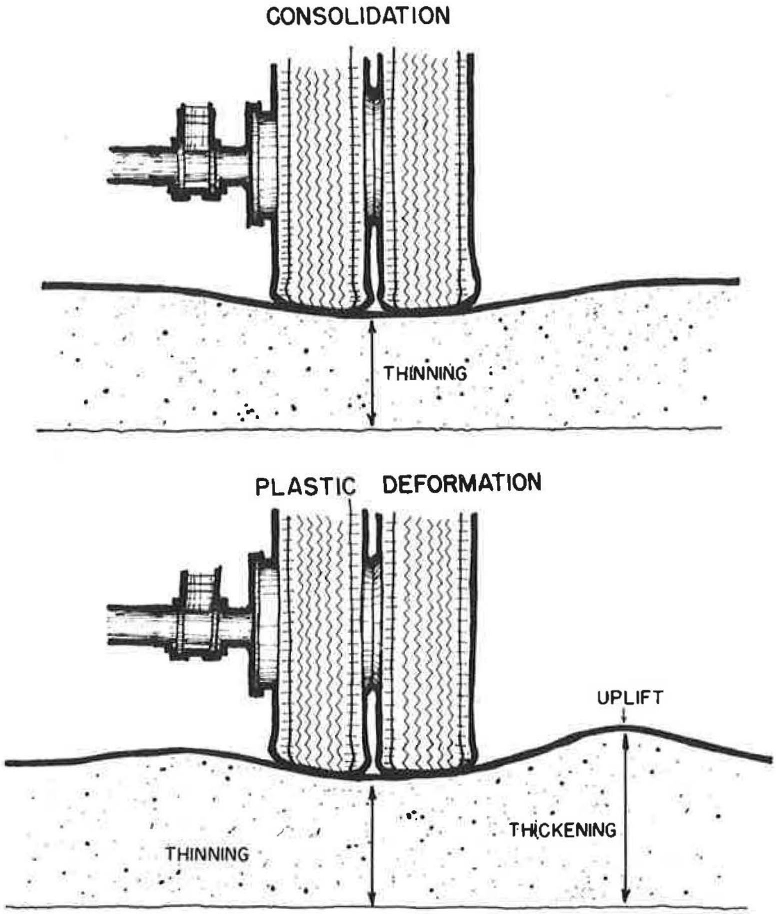


Figure 1. Permanent deformations.

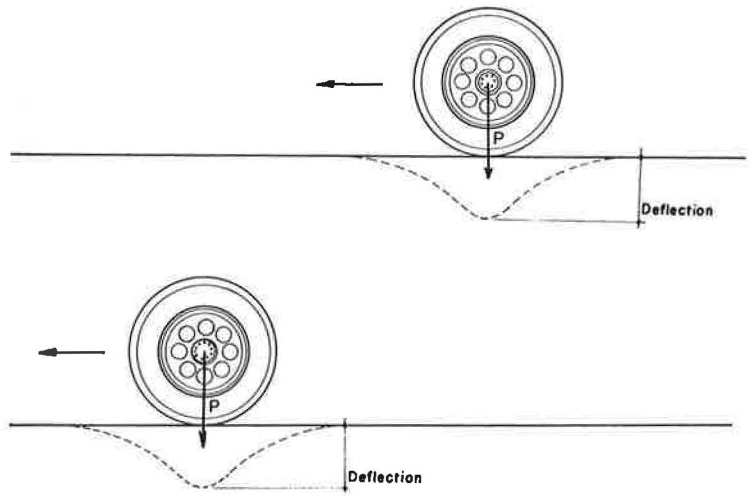


Figure 2. Transient deformation deflections.

Plastic flow of the surfacing material is taken into consideration in the Marshall method of design for bituminous mixture. Plastic ruptures of base, subbase, or subgrade, are foreseen in the CBR method of pavement design. Thus, a well-constructed pavement designed by the CBR method with a bituminous surfacing mixture designed by the Marshall method is secure from plastic deformations.

Expansion. —In addition to consolidation and plastic deformation, pavement may undergo other deformations caused by changes of volume in the material of the subgrade due to moisture variations. Thus a volume increase of expansive soil, produced by water absorption, causes pavement deformations. This increase of volume usually causes a decrease of the subgrade's supporting capacity, which weakens the soil's resistance to shearing stresses, causing rupture of the subgrade.

TRANSIENT DEFORMATIONS—DEFLECTION

A transient deformation is one which disappears when the load producing it is removed. In viscoelastic deformation, a certain time lapse exists between the removal of load and the complete recovery of the deformation. In elastic deformation, the recovery occurs immediately after removal of the load.

Pavement deflections adjust themselves in the latter group. They can be defined as vertical elastic deformations, which occur on pavements due to the passage of wheel loads.

Any pavement which satisfactorily supports the loads for which it was designed will become elastic in a relatively short time, thus preventing permanent deformation. The future performance of such a pavement, therefore, depends on the magnitude of elastic deformations.

Pavements designed by the classical methods, CBR for example, are planned to avoid rupture due to shearing; the results of expansion are taken into consideration by testing the soil after water absorption; the degree of compaction to be obtained during the construction guarantees a reduction of consolidation due to traffic to a minor value. However, the influence of elastic deformations in the future structural performance of the pavement is not taken into consideration.

An analysis of the measurement results of the pavement elastic deformations or deflections shows that excessive deflections cause cracks which destroy the surfacing layer. Even with equipment like the Hveem resiliometer, it is not possible to predict precisely the deflections of a given pavement through laboratory tests made on samples of the subgrade or of the various layers of the pavement. This deflection measurement is taken after the pavement's construction. The data obtained to date show that maximum allowable deflections exist for different types of pavements, above which cracks occur due to fatigue. Deflection measurements have also yielded information concerning the capacity of various materials to distribute the loads to the subjacent layers. These data and information contributed to the methods for pavement overlay design which are discussed later.

DEFLECTION MEASUREMENTS

Various instruments have been used to measure pavement deflections, for example, the electronic measurer, used in California and on the WASHO and AASHO Road Tests. This device consists of a movable coil transformer fixed to the pavement and joined to a recording-amplification system. The passage of vehicle wheels causes a displacement of the movable coil in relation to a fixed nucleus, thus producing a change in the intensity of the electric current. This variation of current is amplified and registered on a graph as a deflection.

Another instrument, used principally in France, is the optical deflectometer which measures the vertical displacement of a luminous point on the pavement.

The instrument which has had worldwide use for pavement deflection measurement is the Benkelman beam, designed by A. C. Benkelman, formerly of the U. S. Bureau of Public Roads, and used for the first time on WASHO Road Test in 1953.

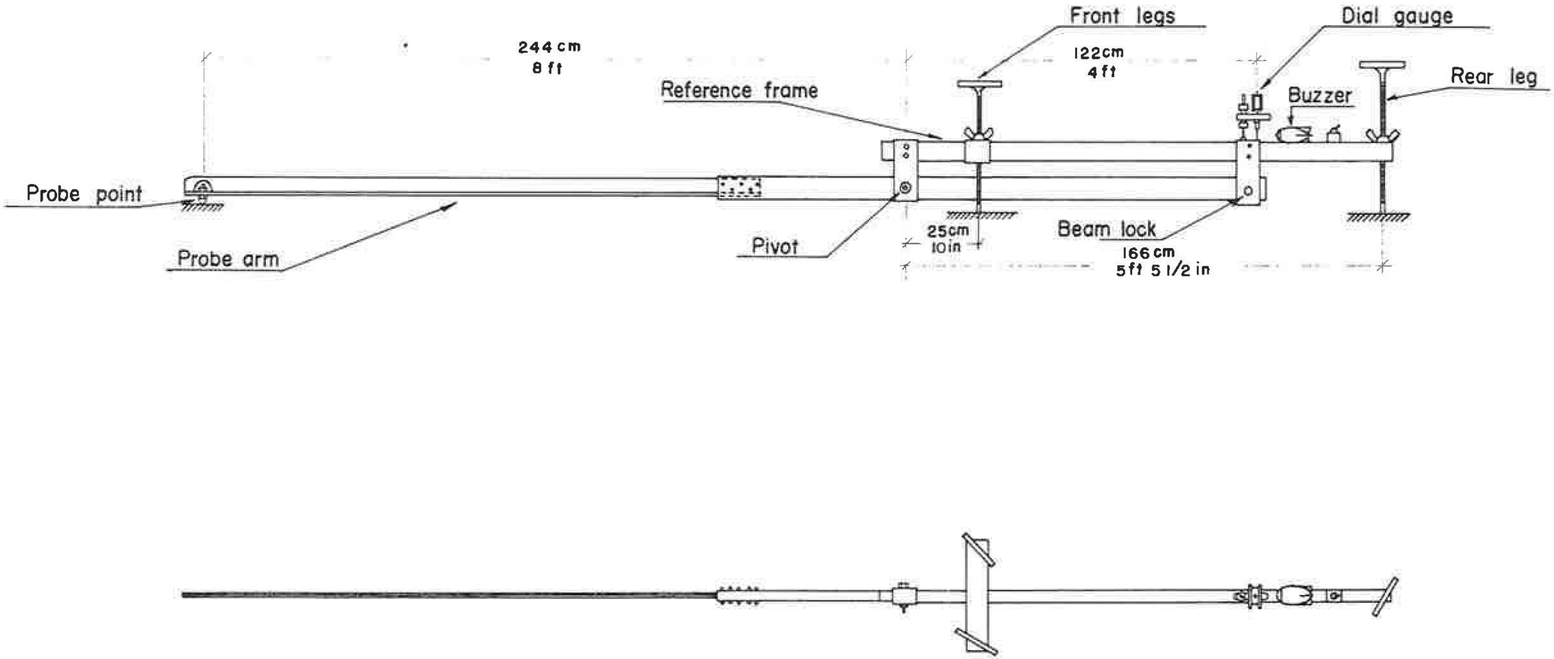


Figure 3. Benkelman beam.

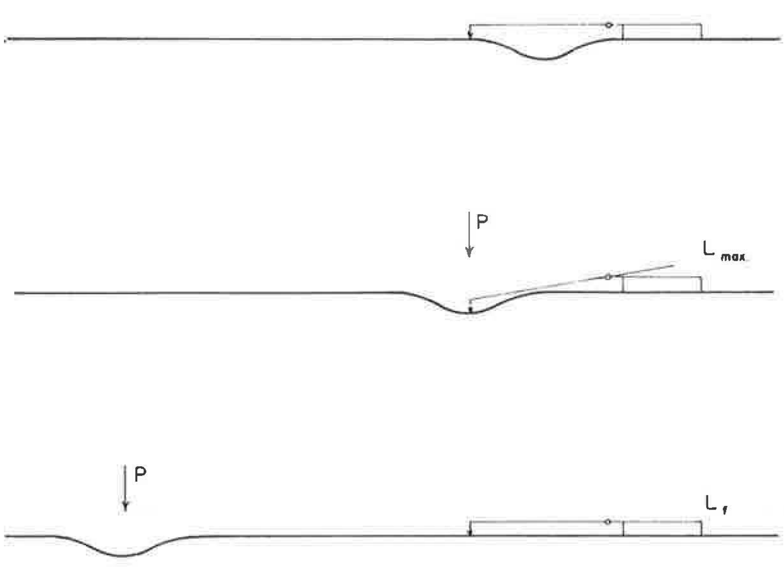
BENKELMAN BEAM

The Benkelman beam is essentially composed of a fixed part and a movable beam (Fig. 3). The fixed part rests on the pavement supported by three adjustable legs. The movable beam is coupled to the fixed part by means of a hinge. One of its ends (toe of probe) remains in contact with the pavement on the spot where the deflection measurement is required. The other end is in contact with a dial gage which indicates any vertical movement of the toe of probe. The fixed part is also equipped with a buzzer which reduces to a minimum the friction of the movable parts during the operation.

Methods of Measurement

WASHO Method. —The original deflection measuring method, using the Benkelman beam, adopted on WASHO Road Test (Fig. 4), is used in essentially the following manner:

1. Insert the movable part of the Benkelman beam between the dual tires of a truck carrying a certain wheel load, resting the toe of probe on the pavement at a distance of approximately 4. 5 ft (1. 35 m) ahead of the truck's rear axle.
2. Switch on the buzzer and make the initial reading (L_o) on the deflectometer.
3. Move the truck slowly forward, making the maximum reading (L_m) when the rear wheels pass the beam's toe of probe.



$$\boxed{D = 2 \times (L_{max} - L_o)}$$

$$\boxed{R = 2 \times (L_f - L_o)}$$

Figure 4. WASHO method of deflection measurement.

4. Make the final reading (L_f) when the truck has passed the toe of probe by at least a distance of 10 ft (3 m).

5. The real deflection (D) is defined as twice the difference between the maximum and initial reading, or

$$D = 2\,(L_m - L_o)$$

6. The residual deflection (D_r) or residue is defined as twice the difference between the final and the initial readings.

$$D_r = 2\,(L_f - L_o)$$

The beam was designed taking into consideration that the area deformed at the moment of the first reading does not reach the toe of probe. It should, therefore, have a radius inferior to 4.5 ft. The residual deflection or residue was considered as permanent deformation or transient deformation of very slow recovery.

Analysis of Cause of Residuals.—The WASHO method continued to be used by the engineers who worked with the Benkelman beam. However, many of them started looking for an explanation for the frequent occurrence of the residual deflections where no permanent pavement should take place by the unique passage of the load of a test truck.

Dunlop and Stark observed (9) that it was not uncommon for the pavements rated as very good to have residual values as high as 0.010 in. On most of these good pavements the average of the ten heaviest daily wheel loads exceeded the test vehicle wheel load by 1,000 to 3,000 lb. Because these pavements had received thousands of the heavier load repetitions, it was reasoned that either the residual values were in error or else they had recovered after some unknown period of time. Otherwise, many of the older pavements would have settled several feet under the action of the traffic.

Results of their tests for several test sites on four different test sections indicate much larger areas of influence than previously reported in literature.

Another conclusion of the study was that all points on the pavement deflect to their full value immediately on application of the load, and rebound almost immediately when the load is removed.

Observations reported by the Canadian Good Roads Association (5) resulted in a modification of the WASHO procedure known as the CGRA test method (see Appendix A).

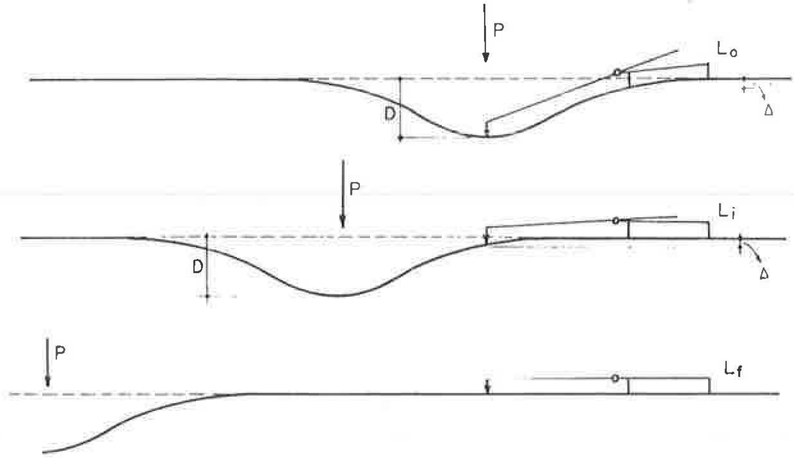
In essence, the CGRA test method determines the rebound characteristics of a pavement subject to a standard 18,000-lb axle load. The dial gage is initially recorded when the probe is between the tires; a second reading is taken when the truck is moved 8 ft 10 in. from the probe, and a final reading with the vehicle at least 30 ft away. Readings are only observed when the pavement rate of movement is less than 0.001 in./min. Apparent rebound measurements can be recognized by comparing the intermediate and final readings. If a differential of more than 0.001 in. exists, the reading is apparent. The apparent measurement can be corrected to determine the actual rebound value by means of the formula in Appendix A.

Canadian Good Roads Association Method.—The formula obtained for correction of the apparent deflection when the intermediate reading does not coincide with the final reading was the following (Fig. 5):

$$D = D_a + K \cdot \Delta$$

where

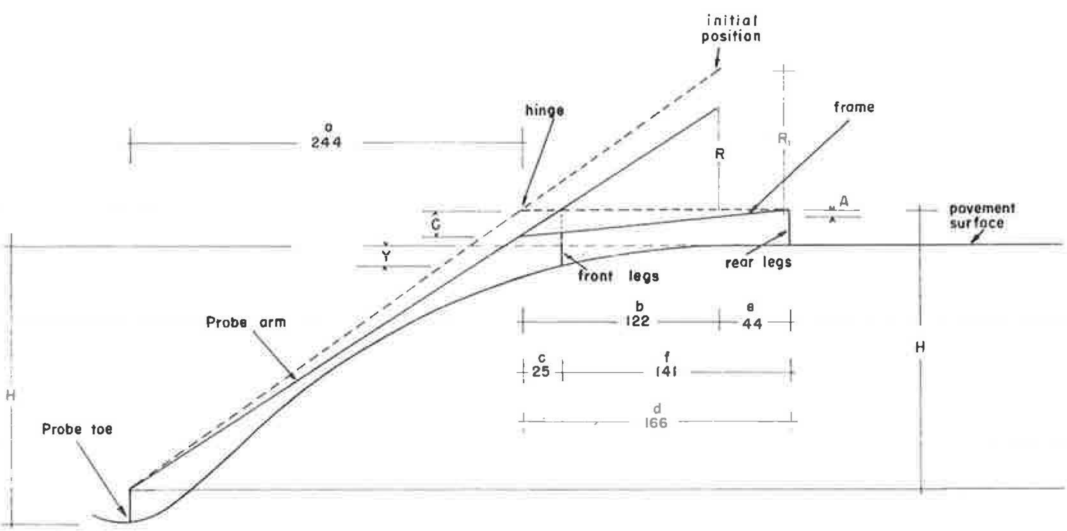
D = true deflection;
D_a = $2\,(L_f - L_o)$ = apparent deflection;
Δ = $2\,(L_f - L_i)$ = vertical displacement of front legs;
L_f = final reading;
L_i = intermediate reading;
L_o = initial reading; and
K = beam constant.



D_a = Deflexão aparente = $2(L_f - L_o)$
Apparent defection
Δ = Deslocamento vertical dos pés dianteiros
Vertical displacement of front legs
$\Delta = 2(L_f - L_i)$

D = Deflexão verdadeira :
True deflection
$\boxed{D = D_a + 2{,}91 \times \Delta}$

Figure 5. CGRA method of deflection measurement.



SYMBOLS

H - Height of datum
R - Apparent reading
Δ - Settlement of front legs
Obs. - Measures in cm
A - Vertical movement of datum at dial gauge
C - Vertical movement of hinge
R_1 - True reading

Figure 6. Benkelman beam in initial position.

The beam constant can be easily determined. Figure 6 shows a Benkelman beam in the position corresponding to the initial reading on a spot of the pavement where the deflected area reaches the front legs of the beam and thus produces their vertical displacement.

$$A = \frac{e}{f}\Delta \tag{1}$$

$$C = \frac{d}{f}\Delta \tag{2}$$

$$\frac{H}{R_1} = \frac{a}{b} \tag{3a}$$

$$\frac{H - C}{R + C - A} = \frac{a}{b} \tag{4a}$$

As a = 2b, we have

$$H = 2R_1 \tag{3b}$$

$$H - C = 2R + 2C - 2A$$

Therefore,

$$H = 2R + 3C - 2A \tag{4b}$$

Substituting A, C and H in Eq. 4b by their values given by Eqs. 1, 2 and 3b gives the following:

$$2\,R_1 = D = \text{true deflection; and}$$

$$2\,R = D_a = \text{apparent deflection}$$

where

$$D = D_a + K \cdot \Delta$$

$$K = \frac{3d - 2e}{f}$$

Deflection measurement using the Benkelman beam—CGRA method, initial position.

Pavement fatigue failure—alligator cracks on both inner and outer wheelpaths.

Alligator cracks and hole formation.

Alligator cracks and holes spreading along pavement.

For the beam with the dimension shown in Figure 3,

$$K = \frac{3 \times 166 - 2 \times 44}{141} = 2.91$$

The true deflection can therefore be calculated by the expression:

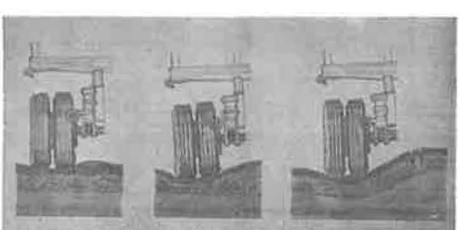
$$D = D_a + 2.91\,\Delta$$

For example, determine the true deflection from the following data obtained during a deflection measurement:

L_o = initial reading = 5.00 mm,
L_i = intermediate reading = 5.47 mm, and
L_f = final reading = 5.50 mm.

The calculations are

D_a = 2 (5.50 - 5.00) = 1.00 mm, and
Δ = 2 (5.50 - 5.47) = 0.06 mm.



Plastic failure of flexible pavement.

Plastic failure of asphalt surfacing—unstable bituminous mixture.

Plastic failure of base and subbase—unstable base and subbase materials.

Plastic failure of subgrade—poor foundation, underdesigned pavement.

Therefore

$$D = 1.00 + 2.91 \times 0.06 = 1.17 \text{ mm}$$

$$\text{True deflection} = D = 1.17 \text{ mm}$$

INFLUENCE OF SURFACING TEMPERATURE ON DEFLECTION MEASUREMENTS

The stiffness of bituminous surfacing varies greatly with temperature. As the temperature decreases the bitumen becomes more viscous and the surfacing stiffens, thus increasing the load-spreading capacity of the pavement structure and decreasing the deflection value.

In addition to variation in stiffness with temperature, the surfacing is squeezed between the tires of the double wheels during load application, and undergoes a certain deformation which is also influenced by temperature and which is composed of two parts: a transient viscoelastic type of deformation which returns to normal after the load has been taken away and a permanent plastic deformation.

This surfacing distortion, which occurs mainly at high temperature, influences deflection measurements when the WASHO method is used, but can be disregarded when the rebound deflection is measured by the CGRA method.

Tests made by Pereira, in Paraná, and by the author, in Guanabara, show however, that temperatures higher than 20 C do not influence deflection measurements. In Brazil, where the bituminous surfacing is almost always above 20 C, we do not correct deflection due to temperature.

CHARACTERISTIC DEFLECTION

When studying the structural condition of a pavement in service for the purpose of planning maintenance, or for overlay design, sections or areas of the pavement must be considered and not isolated points.

In the case of overlay, for construction reasons the thickness of the new layer must be constant along a determined length.

Even if the deflection or the structural strength of the former pavement is determined at 1-ft intervals, it would not be practical to vary the overlay thickness at each foot, following all the strength variations of the pavement.

Pavement strength varies from point to point, in the manner of deflections, because of the variation of the subgrade soil properties and the material composing the different layers of pavements, and also as a result of the lack of uniformity in the construction method. It is, therefore, important to establish a deflection which will best represent each pavement section or area.

A study of pavement deflections has shown that deflections measured along a section present a normal or Gauss distribution. Thus, if we know a certain number (n) of deflections (D) measured along a section of pavement, assuming that they are distributed according to the Gauss law, we could calculate the mean deflection ($\bar{D}$), the standard deviation (σ), and the coefficient of variation (v), using the following expressions:

$$\bar{D} = \frac{\sum D}{n}$$

$$\sigma = \sqrt{\frac{\sum (D - \bar{D})^2}{n - 1}}$$

$$v = \frac{\sigma}{\bar{D}}$$

D varies from point to point; σ and v give an indication of this variation, which is greater in sections where there is a larger dispersion of deflection values.

The standard deviation gives an idea of dispersion when the mean is the same; the coefficient of variation indicates minor or major value dispersion even for sections with different mean deflections.

Following the normally adopted criteria in statistical analysis, we can establish as characteristic deflection of a given section, i. e., the deflection that best represents the section, the value:

$$D_c = \bar{D} + t\sigma$$

Considering that each mean deflection represents a certain extent (or area) of pavement, each value of t corresponds to a percentage of the total extent, or area, with probabilities of presenting deflections superior to the characteristic deflection D_c as indicated in the following table:

Characteristic Deflection ($D_c = \bar{D} + t\sigma$)	Pavement Extent or Area with $D > D_c$ (%)
$D_c = \bar{D}$	50
$D_c = \bar{D} + \sigma$	15
$D_c = \bar{D} + 1.3\sigma$	10
$D_c = \bar{D} + 1.65\sigma$	5
$D_c = \bar{D} + 2\sigma$	2
$D_c = \bar{D} + 3\sigma$	0.1

In the case of overlay design, the overlay thickness must be such that only a small area of the section is underdesigned. It would not be logical, for instance, to design an overlay considering the mean deflection as the characteristic one; i. e., $D_c = \bar{D}$, because in this case 50 percent of the overlay area would probably be underdesigned, and the new layer would show distress shortly after the section is opened to traffic. By taking $D_c = \bar{D} + 3\sigma$ for characteristic deflection, we could practically insure prevention of distress due to underdesigning, but in this case the thickness would be exaggerated and the project too expensive.

Thus, to avoid the two foregoing problems we should adopt an intermediate value between $\bar{D}$ and $\bar{D} + 3\sigma$ as characteristic deflection of the section.

Taking as characteristic deflection that deflection which is exceeded only by the deflections occurring in a small pavement area, we guarantee that only a small area of overlay will be subject to distresses and, because such a characteristic deflection generally has a value close to the highest deflections in the section, distress will not appear until some time after the section is opened to traffic. Logically, it is more economical to strengthen these few points some years later than to use an exaggeratedly thick overlay at the start.

Wilkins and Campbell (32) recommend performing at least 10 determinations for each 1,000-ft (300-m) section, and taking as characteristic deflection for overlay design or for planning maintenance the value:

$$D_c = \bar{D} + 2\sigma$$

Thus only approximately 2 percent of the overlay would be underdesigned and subject to distresses.

Ruiz (27) recommends performing at least 30 determinations for each section and taking $D_c = \bar{D} + 1.65\sigma$ as characteristic deflection. This value gives a 5 percent chance of having an underdesigned overlay.

Following the Canadians' recommendation, we have adopted the value $D_c = \bar{D} + 2\sigma$ as characteristic deflection in overlay projects.

CRITERIA FOR SELECTION OF POINTS FOR DEFLECTION MEASUREMENTS

The literature shows great diversity of opinion concerning the choice of points where deflection measurements are to be performed.

1. Zube and Bridges (34) make the following recommendations for sections with two traffic lanes. On two-lane roads less than 3 mi long, deflections are made at intervals of 50 ft in the lane judged to be representative of the most seriously distressed. Sometimes one-half the project is tested in one lane and the other half in the opposite lane. Using one beam, the deflection tests are alternated, with twice as many in the right or outer wheelpath as in the left or inner wheelpath. During the deflection testing phase a note is made at each test point regarding whether the pavement is cracked or not within 2 to 5 ft of the test point. During the data analysis phase these notes help to establish the reasons for differences in deflections between cracked and uncracked areas. On projects longer than 3 miles test sections 1,000 ft long are selected in each mile of pavement and an analysis similar to the foregoing is made. The data are evaluated on the basis of each mile of pavement.
2. The Special Committee on Pavement Design and Evaluation of the Canadian Good Roads Association (5) suggests that 10 random rebound deflection measurements be made. For flexible pavements the test points are located on an outer wheelpath, 3 ft from the edge of a 12-ft traffic lane. The stations of the test points are selected from a table of random numbers.
3. Welsh (33) recommends that deflection determinations for city streets be made on the outer wheelpaths of both lanes, at 50-ft (15 m) intervals.
4. We recommend that a section be divided into subsections every 1,640 ft (500 m) and the characteristic deflection (D_c) be determined for each subsection. This is suggested for studies made in Brazil, to obtain data for planning maintenance, or for overlay projects.

The deflections should be measured on the outer and inner wheelpaths alternately on both lanes, at 164-ft (50-m) intervals (Fig. 7).

SEASONAL DEFLECTION VARIATION

We have seen that deflections vary from point to point along a section, and have examined the criteria to determine most accurately the characteristic deflection of the section in question. In addition to that kind of variation, there is also seasonal variation.

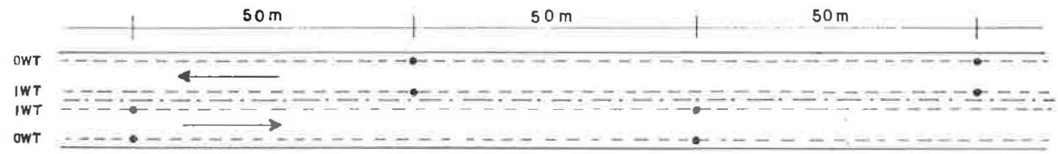


Figure 7. Location of deflection measurement points on highway section.

The bearing capacity and the deflection of a pavement are closely related to the subgrade's performance. Because the bearing capacity of the subgrade soil constituents varies with the soil moisture content, and this variation takes place throughout the year, pavement deflection is not constant and depends on the season when it is determined. During the months when the subgrade soil is more saturated, deflection is greater and vice versa.

In countries subject to freeze and thaw, the subgrade has a maximum moisture content in the spring producing maximum deflection. Figure 8 shows the deflection variation throughout the year of a pavement section in Canada. The maximum deflection is during the month of April when thaw sets in.

As it is not possible to measure all deflections during the same period of the year, the Canadians recommend that when measurements are taken during periods not representing maximum deflection, the latter value be calculated. This calculation can be used in pavement overlay design or as a means of comparing the structural condition of pavements in service, to plan maintenance.

Using data obtained in plate bearing tests and deflection measurements using the Benkelman beam, the Canadians determined the relationship between the loss of a pavement's strength and its deflection variation (Fig. 9).

Putting the value of the characteristic deflection of a section for a given period of the year on the abscissa and the value of characteristic deflection of the same section, but of another period of the year, on the ordinate, the percentage of strength loss suffered by the pavement can be determined. Two examples follow.

1. If the characteristic maximum deflection of a pavement section as measured in April (thaw period) is 0.100 in. (2.54 mm) and the deflection determined in July is 0.042 in. (1.07 mm), determine the strength loss suffered by the pavement from July to April. This value is found by plotting 0.042 in. on the abscissa of Figure 9, and 0.100 in. on the ordinate; the strength loss is R = 50.

2. If the strength loss of a pavement between July and April (maximum deflection period) is 50 percent, calculate the characteristic maximum deflection of the pavement

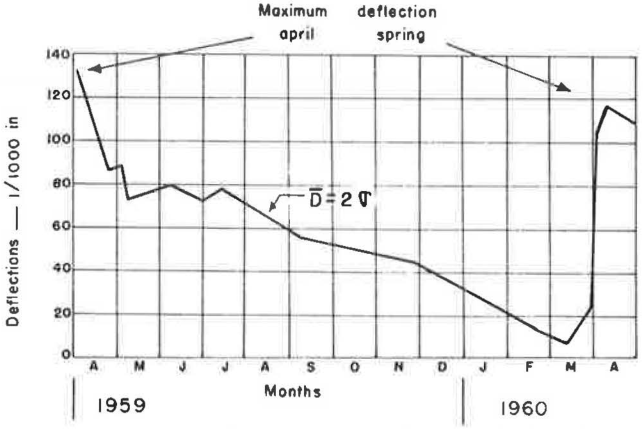


Figure 8. Seasonal variation in deflection for Canada.

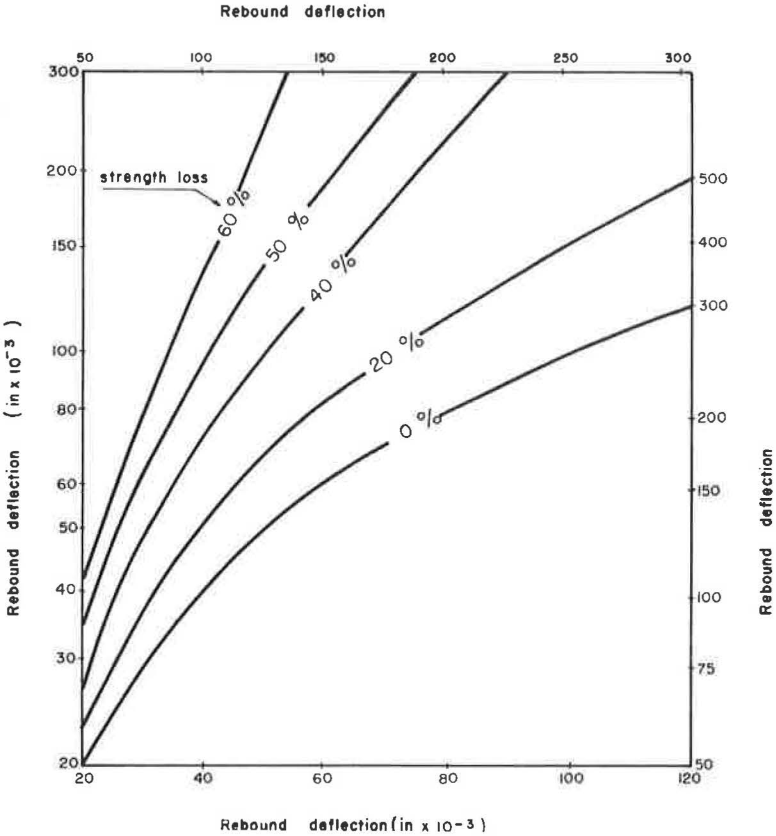


Figure 9. Relation of CGRA Benkelman deflections for various pavement strength losses.

in a section where the characteristic deflection determined in July is 0.024 in. The solution is found by plotting D_{July} = 0.024 in. on the abscissa and R = 50 percent on the curves indicating strength loss. The value of maximum deflection (occurring in April) on the ordinate is D_{max} = 0.046 in.

DEFLECTION AND PERFORMANCE OF FLEXIBLE PAVEMENTS

The study of pavement deflections by WASHO, where the Benkelman beam was first used, and later by AASHO, showed that the performance and life of flexible pavements with bituminous surfacing are closely related to the pavement deflections caused by the passage of vehicle wheels.

An analysis of various performances of pavement sections subject to the same traffic showed that the sections with greater deflection had the worst performances and the least useful lives. The sections with low deflection needed little maintenance and possessed a longer period of life.

After the results obtained on American road tests became known, various engineers and highway organizations studied pavement deflections using the Benkelman beam as a measuring instrument.

A large number of studies and conclusions concerning the influence of deflection on pavement performance have been published (1-34). For example, the Road Research Laboratory of England reported the following conclusions.

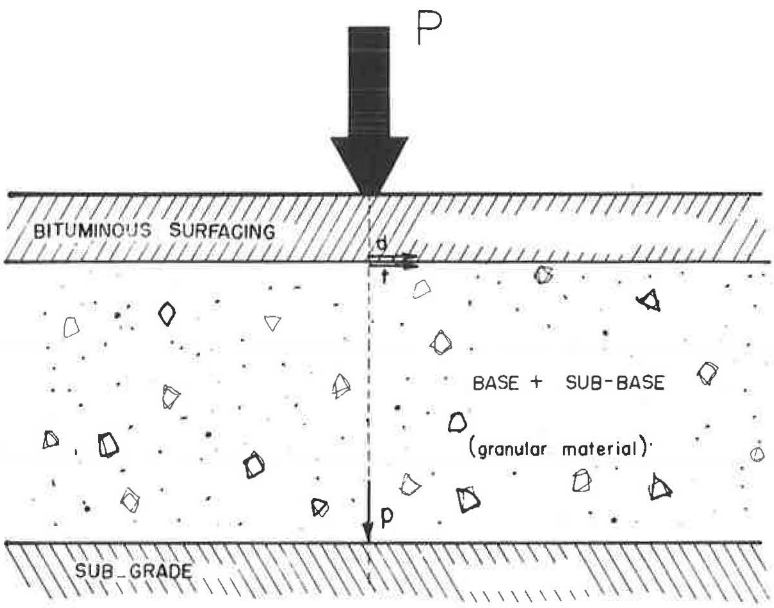
1. Rebound deflection measurements are of great use in the investigation of pavement characteristics;

2. The Benkelman beam allows pavement uniformity verification in a relatively fast and simple way;
3. With the data now being obtained, it will probably be possible to estimate future pavement performances using the Benkelman beam for measuring deflection;
4. The main performance characteristic of all British test sections was the notably superior performance (fewer deflections, fewer distresses, greater life) of pavements surfaced with asphaltic-concrete in comparison to those surfaced with bituminous-macadam;
5. Critical deflection is approximately the same for crushed stone and for gravel pavements;
6. In subgrades of the same condition the structure's stiffness increases (deflection decreases) with the thickness of the pavement;
7. Crushed stone is more efficient than gravel in reducing deflections; and
8. Bituminous-macadam contributes more than granular bases to pavement stiffness.

CRITICAL STRESSES ON FLEXIBLE PAVEMENTS

To explain further the relationship between deflection and pavement performance, the following paragraphs examine the critical stresses to which the various layers of the structure are subject, as well as the life phases of a flexible pavement with bituminous surfacing.

Figure 10 shows a pavement made of bituminous surfacing, granular base and sub-base constructed on a homogeneous subgrade. Due to the load produced by vehicle wheel passage represented by P, the structure is deformed, producing a tensile stress (t) beneath the wheels corresponding to a strain (d) on the underside (distended fibres)



t = tension stress at the surfacing bottom.
$t > \sigma_t$ → Cracks on the surfacing.
Solution: $D < D_{adm}$
p = vertical pressure on the subgrade.
$p > p_{adm}$ → Plastic rupture of the subgrade.
Solution: $CBR > CBR_{min}$

Figure 10. Critical stresses (t and p) on pavements.

of the surfacing. Assuming that the material of base and subbase is of good quality, and that no shearing rupture takes place, P produces a pressure (p) on the subgrade.

When p is kept below a determined allowable pressure (p_{adm}), the subgrade does not undergo shearing rupture, nor does plastic deformation take place. To insure this condition, the structure must have an equal or a higher thickness than that indicated, for example, by the subgrade's CBR.

It is necessary to keep the pavement deflection (D) below a certain value (D_{adm}) to prevent surfacing cracks. This prevents t which corresponds to d on the underside of the surfacing from going beyond a certain value above which the bituminous material ruptures due to fatigue.

LIFE PHASES OF FLEXIBLE PAVEMENT

This section examines the performance of pavements which are well designed by the classical method (i. e., CBR), having no plastic rupture in the subgrade or in any layer of the structure. Underdesigned pavements have a relatively short life, being subject to rupture due to shearing by the passage of a small number of the heaviest wheel loads.

The life of a well designed flexible pavement can be divided into distinctive phases.

Consolidation Phase

During the initial phase of pavement life the various layers of the structure undergo some consolidation due to the action of load transmitted by the vehicle wheels. This phase is relatively short and the magnitude of consolidation depends on the compaction received by the various layers during the pavement's construction. Ruts appear in wheelpaths, but without any cracks.

Elastic Phase

Soon after the consolidation phase during which each load produces a permanent deformation (consolidation) which tends to stabilization, apart from an elastic deformation (deflection), the structure enters its second phase during which a genuine elastic performance occurs. Each wheel load produces a deflection which recovers after the vehicle's passage.

The pavement's life depends mainly on how long this second phase lasts, because it is closely related to the occurrence of deflections. A study of deflections of pavements in service has shown that pavements with low deflection have long lives as opposed to those with high deflection.

Fatigue

Fatigue corresponds to the final phase. Elastic deformations (deflections) caused by the wheel loads produce tensile stresses on the bituminous surfacing, which after a period of time fails due to fatigue.

Fatigue failure starts with longitudinal cracks on the underside of the surfacing under the wheelpaths and the surrounding surface. As the action continues, traffic produces transverse cracks which soon become alligator cracks. Cracks facilitate the penetration of surficial waters to the pavement's interior, thus weakening the whole structure. As the surfacing deteriorates, holes form which spread and destroy large areas of pavement—generally at the end of rainy seasons. The pavement thus comes to the end of its useful life.

ALLOWABLE DEFLECTIONS

As discussed previously, if cracks are to be avoided pavement deflection must be kept below a critical value which depends on the traffic and on the type of pavement. Although the critical and allowable deflections to which heavy traffic pavements are subject have been reported, literature relating such deflections to traffic characteristics is scarce.

The following paragraphs describe several authors' observations for heavy traffic pavements.

1. Ruiz (27). Argentine pavements subjected to heavy traffic crack when deflection is higher than 0.90 mm (determined by a truck with 18,000 lb—8.2 ton—axle load and tire pressure of 80 psi). Also, $D_{adm} = 70$ to $50 \frac{1}{100}$ mm are the allowable deflection values for pavements subjected to heavy traffic.

2. Aratangy (2). Three types of pavement are classified according to deflections (determined for an axle load of 12 tons) as follows:

$$\text{Pavement in good condition} = D < 50 \frac{1}{100} \text{ mm;}$$

$$\text{Pavement in fair condition} = 50 < D < 100 \frac{1}{100} \text{ mm; and}$$

$$\text{Pavement in bad condition} = D > 100 \frac{1}{100} \text{ mm.}$$

3. Lassale and Langumier (16). Adoption of $D_{adm} = 80$ to $50 \frac{1}{100}$ mm as allowable deflection determined for an axle load of 13 tons is recommended.

4. Hveem (14). The following values for allowable deflections determined for an axle load of 15,000 lb (6.8 tons) are recommended.

Pavement Type	Allowable Deflection	
	$\frac{1}{1000}$ In.	$\frac{1}{100}$ Mm
4 in. of asph. concrete on gran. base	17	43
3 in. of asph. concrete on gran. base	20	50
2 in. of asph. concrete on gran. base	25	63
Cement-treated base with bitum. surfacing	12	30
Surface treatment	50	127

5. Road Research Laboratory (18, 19, 20, 26, 28). Allowable deflection recommendations determined for an axle load of 14,000 lb (6.4 tons) are as follows:

a. Granular-base pavements (crushed stone or gravel) with bituminous surfacing (asphaltic concrete or bituminous macadam):

$$D_{adm} = 20 \text{ to } 30 \frac{1}{1000} \text{ in. } (50 \text{ to } 75 \frac{1}{100} \text{ mm})$$

b. Cement-treated base pavements (soil-cement or lean concrete) with bituminous surfacing (asphaltic concrete or bituminous macadam):

$$D_{adm} = 5 \text{ to } 12 \frac{1}{1000} \text{ in. } (13 \text{ to } 30 \frac{1}{100} \text{ mm})$$

Most authors indicate 50 to 80 $\frac{1}{100}$ mm as allowable deflections for flexible pavement (granular base with bituminous surfacing) subject to heavy traffic.

We have adopted the following deflections in our surfacing designs.

$D_{adm} = 50 \frac{1}{100}$ mm—for heavy traffic highways (BR-3, BR-4 and BR-28)

$D_{adm} = 70 \frac{1}{100}$ mm—for medium to heavy traffic highways (RJ-5, RJ-58 and PA-25)

For Brazilian heavy traffic highways, for example the Rio de Janeiro-São Paulo highway, an allowable deflection less than 50 $\frac{1}{100}$ mm may have to be adopted. For very light traffic highways, an allowable deflection of more than 70 $\frac{1}{100}$ mm can be adopted.

In addition to the allowable deflection recommendations for flexible pavements subjected to heavy traffic, other observations have been made correlating allowable deflection with traffic.

Welsch (33), for example, presents a graph (Fig. 11) showing a critical deflection curve (deflection of sections with fine cracks) and an allowable or design deflection curve both in relation to traffic on municipal streets.

The most detailed papers on allowable deflection variation with traffic are those of the Road Research Laboratory (18, 19, 20), which describe a deflection study on experimental road sections using different types of pavements. Deflections of various sections and at various periods of time were determined from the date the pavements were constructed. Together with the deflection measurements, the pavement condition and the cumulative traffic (from inauguration date to the date deflection was determined) was observed. A relationship between allowable deflection (below which the pavement did not present cracks or exaggerated deformations) and cumulative traffic (up to the observation date) was thus obtained.

Figure 12 shows relationships between allowable deflections and traffic for various pavement types.

Taking as an example the curve corresponding to an asphaltic concrete surfacing pavement with a gravel base constructed on a clay subgrade, the allowable deflection

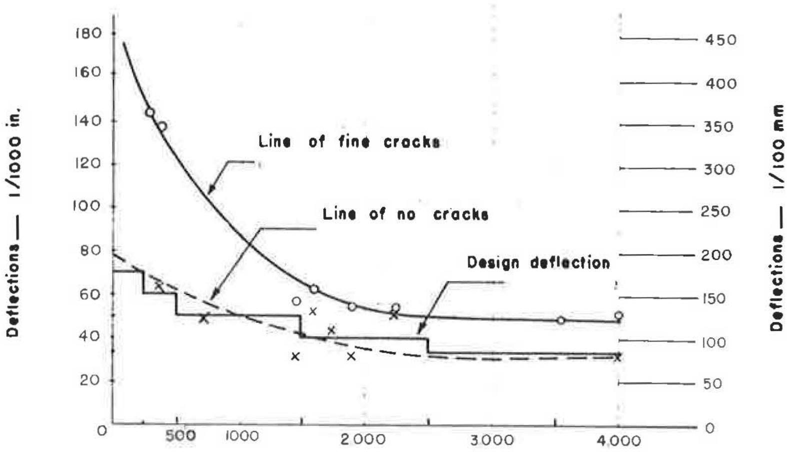


Figure 11. Design deflections for municipal streets of various traffic densities.

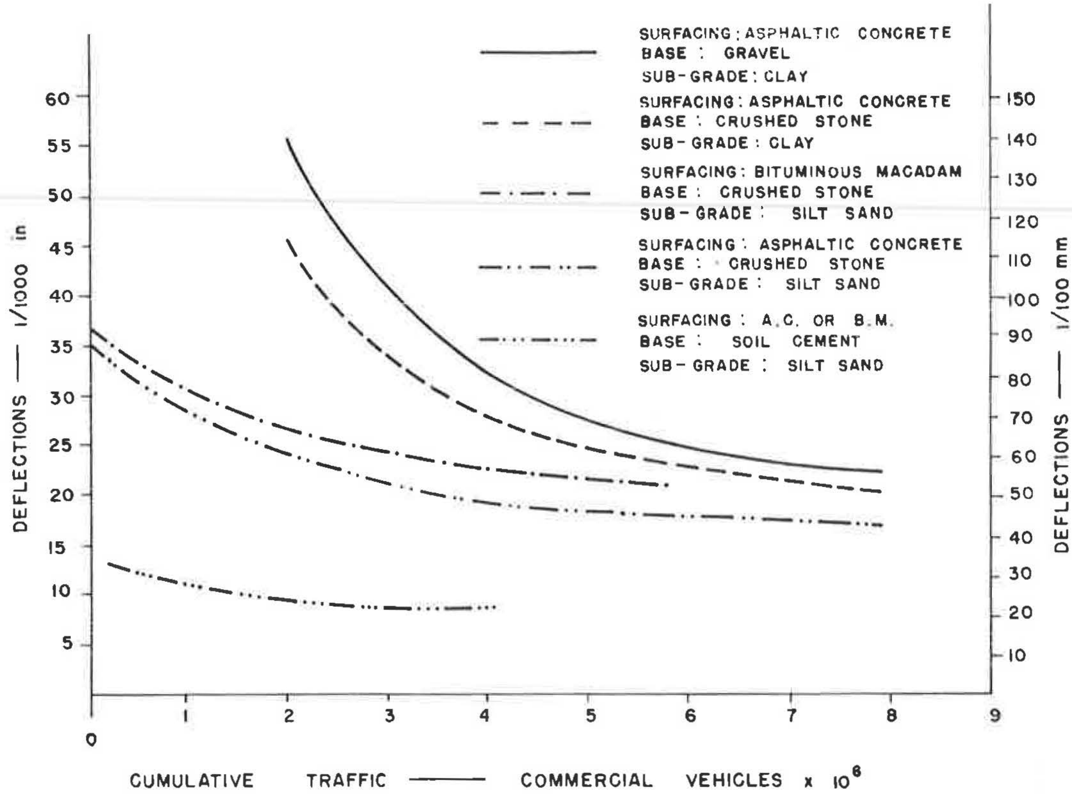


Figure 12. Relation between deflection criteria and cumulative traffic for various types of pavement and subgrade.

for a cumulative traffic of 3×10^6 commercial vehicles is $100 \frac{1}{100}$ mm. For a cumulative traffic of 8×10^6 commercial vehicles, the allowable deflection is reduced to $55 \frac{1}{100}$ mm.

MAINTENANCE AND PAVEMENT DEFLECTION RELATIONSHIP

Due to the sharp increase in traffic volume and wheel loads, most Brazilian highway pavements are cracking and deteriorating rapidly; most are underdesigned for present traffic or were constructed with inadequate material to support high tire pressures and the heavy wheel loads of modern trucks used in highway transportation.

Initially undiscernible surface cracks later expand rapidly into alligator cracks. With the continuation of the traffic action, and generally after intensive rainfall, the first holes appear and soon spread along the pavement, destroying it completely.

In most cases, the maintenance crew only begins to work after the first holes have appeared. The holes are cleaned and filled with cold-laid bituminous mixtures to maintain traffic along the damaged section. As years go by, the holes increase in number during the rainy seasons, and the maintenance cost increases proportionately.

Finally maintenance work becomes impractical because of cost and an overlay is required on the damaged section. At this point the engineer in charge of maintenance must decide what the overlay thickness should be.

Without time or means for more detailed studies, the Brazilian engineer has no alternative other than to determine the overlay thickness arbitrarily basing his judgment on his own experience, on superficial observation of the pavement, and on the opinion of other engineers. Unfortunately, therefore, the paved highway maintenance situation is practically without scheduling or planning.

Deflection measurements using the Benkelman beam can provide the maintenance engineer with the knowledge of pavement structural conditions indispensable to planning maintenance and designing overlay thickness.

Without accurate knowledge pavement overlay design, maintenance, and planning are subject to error and serious loss.

The type of superficial examination currently performed on pavements can only reveal distresses already in progress. Prediction of failures, until a few years ago was very difficult even in countries where highway technology was most highly developed.

The Benkelman beam measures deflections in a relatively simple and fast way. Knowing the characteristic deflection of a pavement area or section and its allowable deflection, the maintenance engineer can solve various problems.

Maintenance Prediction and Planning

By determining the Benkelman deflections along the pavement of a highway system and establishing an allowable deflection (below which no fatigue failure occurs) for each section, the engineer can decide which roads or sections require the most maintenance and schedule preventive maintenance accordingly.

Delimitation of Weak Areas

Determination of deflections along the critical sections of a road presenting the greatest maintenance problems permits delimitation of the areas of highest deflections, corresponding to the weakest pavement areas, such as plastic ruptures. For example, deflections measured on a plastic rupture are two to five times greater than those close to, but outside of, the failed areas.

Drainage

The necessity for, or efficiency of, underdrainage on cuts can be established by determining the deflections along the cut and on the adjacent fills after both the rainy and dry seasons. If the cuts' deflections are much higher after the rainy season than after the dry season in comparison to the fills' deflection variation, deep drainage is necessary.

Studies performed in Brazil have verified that cuts with deficient underdrainage always have high deflections after the rainy seasons and are normally pavement areas where the greatest number of distresses occur.

Influence of Shoulder Types

By comparing deflection values measured in the outer and inner wheelpaths, the engineer can verify the type of influence exerted by the shoulder on the pavement's structural performance.

Aratangy (2) concluded that grassy shoulders adversely affect performance of pavement edges, causing high deflections after the rainy seasons. This is probably due to the greater water retention of grassy shoulders. Shoulder-treated pavements (soil-cement or soil-bitumen) performed best, with a very slight difference between the outer and inner wheelpath deflection.

OVERLAY DESIGN

One of the main applications of data furnished by the Benkelman beam is the designing and planning of pavement strengthening or overlay design of pavements to be restored. Modern methods based on Benkelman deflections have revolutionized the purely empirical overlay design processes used until recently. In the past, the overlay thickness, generally constant along all the section, was determined by the maintenance engineer on the basis of his own experience or superficial observations. This resulted in overdesigned or underdesigned sections with cracks and consequent failures of the latter. It is more practical and economical to vary the overlay thickness along the section in accordance with the structural condition of the former pavement.

With the application of the modern methods based on the deflection of the pavement to be strengthened, the necessary thickness can be determined according to the characteristic deflection of each section.

Deflection studies (5, 27, 34) show that when a pavement is strengthened, a reduction of deflection occurs, depending on the material employed in the overlay and on the pavement's former deflection.

It is not possible to characterize a particular strengthening material by a constant factor of deflection reduction (27) because the overlay's contribution to the deflection reduction is determined by the amount of the original deflection, e.g., if an overlay thickness reduces the original deflection from $D_o = 100 \frac{1}{100}$ mm to $D_h = 50 \frac{1}{100}$ mm, a thickness higher than h is necessary to reduce another section of the pavement presenting a deflection of $D' = 40$ to $D'_h = 20$.

Studying the law of deflection variation with overlay thickness Lassale and Langumier (16) in France and Celestino Ruiz (27) in Argentina verified that a linear relationship exists between overlay thickness and deflection logarithm. Figure 13 shows this for various overlay thicknesses investigated on a section of an Argentine highway (27)

One of the main applications of overlay is to strengthen the structure of old pavements, i.e., to reduce deflections to prolong pavement life. As the deflection must be maintained below a certain value after the overlay construction to prevent cracking of the new layer due to fatigue, the problem of overlay design is to calculate the necessary overlay thickness (h) capable of reducing the former pavement's deflection (D_o) to an allowable value (D_h).

Ruiz (27) presents the following expression for calculating the necessary overlay thickness:

$$h = \frac{R}{0.434} \log \frac{D_o}{D_h}$$

where

h = overlay thickness (cm);

D_o = deflection of existent pavement ($\frac{1}{100}$ mm);

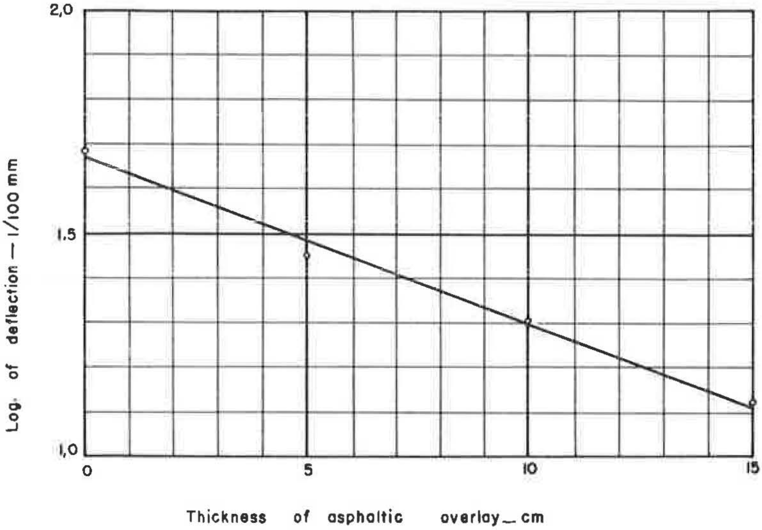


Figure 13. Relation between overlay thickness and logarithm of deflection—Argentina.

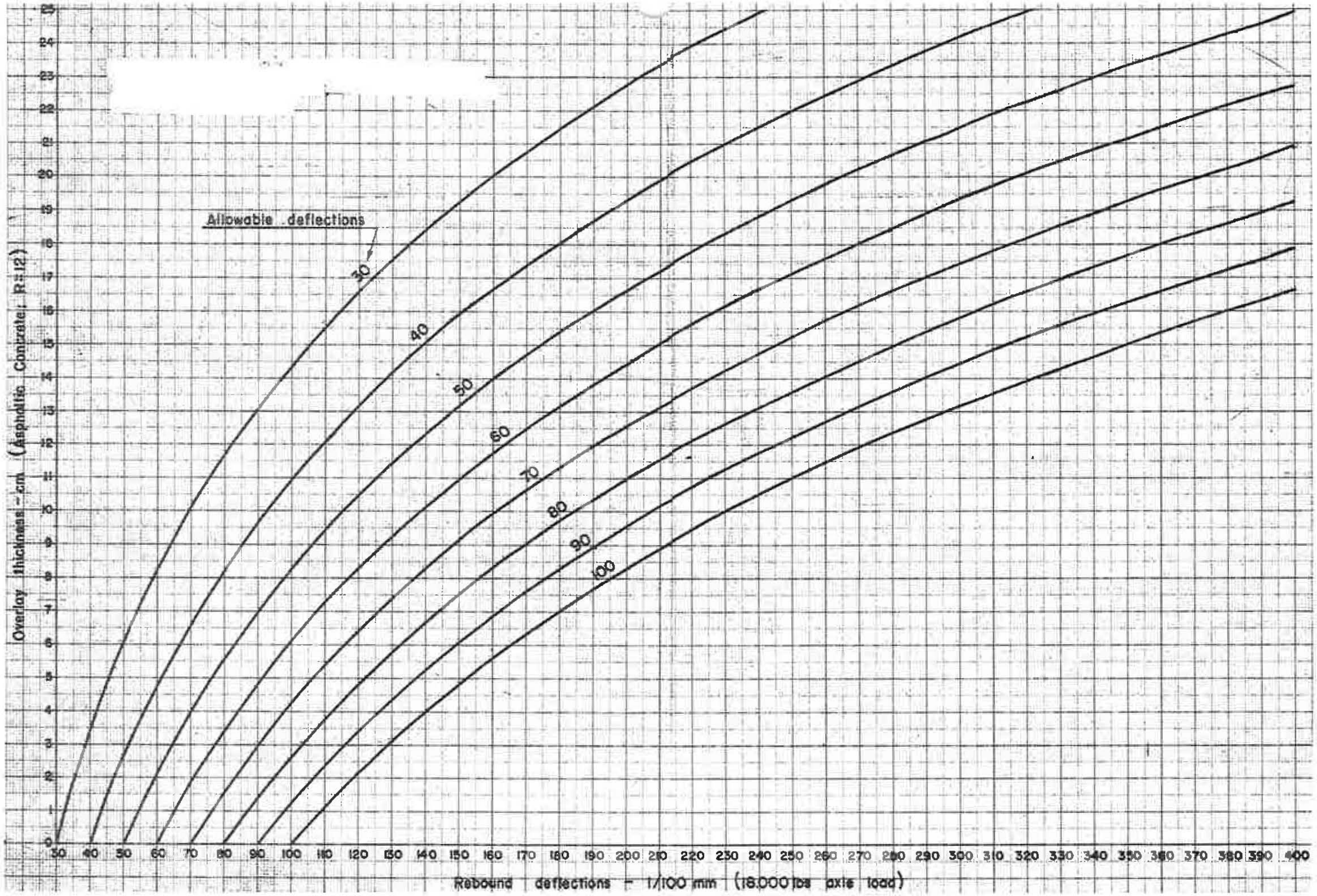


Figure 14. Overlay thickness required to reduce deflections on existing surfaces to various allowable values.

D_h = deflection after overlay construction ($\frac{1}{100}$ mm); and
R = deflection reduction factor, appropriate to material used in overlay (cm).

For asphaltic-concrete, a value of approximately $R = 12$ can be used for calculation of an asphaltic-concrete overlay (Fig. 14).

A more exact value of R for the inherent characteristics of the material adopted in the overlay is easily determined on the job by measuring D_o before and D_h after the construction of an overlay section of known h and calculating the R value by the expression

$$R = \frac{0.434\, h}{\log \frac{D_o}{D_h}}$$

D_h must be determined after at least a month of traffic, so that the new layer is subjected to densification by the action of wheel loads. Between the before and after determinations, no change of the subgrade's condition should occur. If, for example, the first measurement is made after a period of dry weather, and rainfall occurs before the second determination, the calculation of R will be useless. To make sure that the subgrade's conditions do not vary, the deflections for calculating should be made on the inner wheelpath where moisture content variation is less than on the outer wheelpath.

The thickness of the overlay determined by Figure 14 can be easily corrected if R has a value other than 12, using the expression

$$h_R = \frac{R}{12} \cdot h_{12}$$

where

h_R = overlay thickness for material with $R \neq 12$;
R = deflection reduction factor for overlay material ($R \neq 12$);
h_{12} = overlay thickness taking into consideration that $R = 12$; determined from Figure 14 or by the expression

$$h_{12} = \frac{12}{0.434} \log \frac{D_o}{D_h}$$

Overlay must be designed for pavement's worst conditions, i.e., for the period when deflections are highest. The characteristic deflections of each section, determined at a certain period of the year, must be corrected as discussed previously so that the maximum characteristic deflection is obtained. The pavement must be designed to this value.

In Brazil, no reference sections exist, and deflection measurements are taken all year round. If deflection measurements are not made after rainy seasons, which are the periods of minimum strength, we shall have to consider the deflection of a small section as representative of the whole section during this period of maximum deflection.

A study for overlay design can be described as follows.

1. Deflection measurements during a given period.
2. Calculation of characteristic deflection of each subsection of 500 m, for example. Calculation of two characteristic deflections, one corresponding to the outer wheelpath and the other to the inner wheelpath, taking the highest for design. These data are collected for verification of shoulder influence on pavement performance.
3. Calculation of each subsection's maximum characteristic deflection corresponding to the period of the pavement's minimum strength.
4. Calculation of overlay thickness by means of Figure 14, considering $R = 12$.

5. Determination of the real value of R for the material used in the overlay, after its construction has begun.
6. Correction of the thickness determined in paragraph (4) for the material, with $R \neq 12$.

Example

Design the overlay of a given section, taking the following into consideration.

1. The section to be strengthened has the following values in July:

$$\bar{D} = 35 \frac{1}{1000} \text{ in.}$$
$$\sigma = 8 \frac{1}{1000} \text{ in.}$$

2. The reference section has the following values in July and at the period of minimum strength:

$$\bar{D}_{July} = 30; \ D_{max} = 50 \frac{1}{1000} \text{ in.}$$
$$\sigma_{July} = 7; \ \sigma_{max} = 12 \frac{1}{1000} \text{ in.}$$

Solution

1. Using Figure 9 with the deflection in July and the maximum deflection of the reference section, R = 40 percent strength loss.
2. Using Figure 9 with the value of $\bar{D}$ of the section to be strengthened and determined in July, and with R = 40 percent for the strength loss,

$$\bar{D}_{max} = 60 \frac{1}{1000} \text{ in.}$$

3. Since the variation coefficient is maintained approximately the same,

$$\frac{\sigma_{July}}{\bar{D}_{July}} = \frac{8}{35} = 0.23$$

Therefore

$$\sigma_{max} = 60 \times 0.23 = 14$$

4. Thus the characteristic deflection of the section for design purposes is,

$$D_c = \bar{D} + 2\sigma$$

Therefore

$$D_c = 60 + 28 = 88 \frac{1}{1000} \text{ in.} = 224 \frac{1}{100} \text{ mm}$$

5. Using Figure 14 with $D_c = 224$ and adopting an allowable deflection $D_h = 70 \frac{1}{100}$ mm, gives the overlay thickness $\underline{h}$ where $\underline{h} = 14$ cm.
6. Considering the R-value (obtained on the job) to be R = 15 and not R = 12, the corrected thickness is,

$$h_{15} = \frac{15}{12} \times 14 = 17.5 \text{ cm}$$

7. Therefore, the overlay must be 17.5 cm.

DATA NEEDED IN BRAZIL

We have suggested to IPR (Brazilian Highway Research Institute) that a team study, in cooperation with ABP (Brazilian Paving Association) and other highway agencies of the country, obtain Brazilian data which would be of use to those who work with the Benkelman beam.

The following suggestions could have immediate application.

1. Establish reference sections located on different types of pavements and subgrades in various regions of Brazil to determine the period of minimum strength (maximum deflection) and deflection variations throughout the year for each region of the country.
2. Request engineers working with the Benkelman beam to compile data on pavement condition (perfect, fine cracks, alligator cracks); age and type of pavement, estimated traffic; deflection determination data; rainfall; and other information necessary to establish allowable pavement deflections.
3. Determine the R-value proposed by Ruiz for the different materials used in pavement overlay.
4. Determine the deflection variation with temperature for the various types of pavements.
5. Inasmuch as standardization of norms in the use of the Benkelman beam is of maximum importance, we suggest that the Canadian Good Roads Association method be adopted for deflection measurements.

An example of an overlay design of a section of highway BR-28 is given in Appendix B.

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Appendix A

CGRA DEFLECTION TEST PROCEDURE

Scope

This method of test covers a procedure for the determination of the static rebound deflection of a pavement under a standardized axle load, tire size, tire spacing, and tire pressure.

Equipment

The equipment shall include the following:

1. U. S. Bureau of Public Roads type Benkelman beam having the following dimensions:

Dimension	Ft	In.	Cm
(a) Length of probe arm from pivot to probe point	8	0	244
(b) Length of measurement arm from pivot to dial	4	0	122
(c) Distance from pivot to front legs	0	10	25
(d) Distance from pivot to rear leg	5	5½	166
(e) Lateral spacing of front support legs	1	1	33

2. A 5-ton truck is recommended as the reaction. The vehicle shall have an 18,000-lb rear axle load equally distributed on two wheels, each equipped with dual tires. The tires shall be 10.00 × 20, 12-ply, inflated to a pressure of 80 psi. The use of tires with tubes and rib treads is recommended.
3. Tire pressure measuring gage.
4. Thermometer (0-120 F) with 1 deg divisions.
5. A mandrel for making a 1.75-in. deep hole in the pavement for temperature measurement. The diameter of the hole at the surface shall be one-half inch and at the bottom three-eighths of an inch.

Procedure

1. The point on the pavement to be tested is selected and marked. For highways, the points are located at specified distances from the edge of the pavement according to the width of the lane, as follows:

Lane Width (ft)	Distance from Pavement Edge (ft)
9 or less	1.5
10	2.0
11	2.5
12 or more	3.0

2. The dual wheels of the truck are centered above the selected point.
3. The probe of the Benkelman beam is inserted between the duals and placed on the selected point.
4. The locking pin is removed from the beam and the legs adjusted so that the plunger of the beam is in contact with the stem of the dial gage.
5. The dial gage is set at approximately 0.4 in. The initial reading is recorded when the rate of deformation of the pavement is equal to or less than 0.001 in./min, i.e., dial measurement rate is less than 0.0005 in./min.
6. The truck is slowly driven forward a distance of 8 ft 10 in. and stopped.
7. An intermediate reading is recorded when the rate of recovery of the pavement is equal to or less than 0.001 in./min.
8. The truck is driven forward a further 30 ft.
9. The final reading is recorded when the rate of recovery of the pavement is equal to or less than 0.001 in./min.
10. Pavement temperature is recorded at least once every hour, inserting the thermometer in the standard hole and filling up the hole with water. At the same time the air temperature is recorded.
11. The tire pressure is checked at 2- to 3-hr intervals during the day and adjusted to the standard if necessary.

Calculations

1. Subtract the final dial reading from the initial dial reading. Subtract the intermediate dial reading from the initial dial reading.
2. If the differential readings obtained compare within 0.001 in., the actual pavement movement is twice the final differential reading.
3. If the differential readings obtained do not compare to 0.001 in., twice the final differential dial reading represents the apparent pavement deflection.
4. Apparent deflections are corrected by means of the following formula:

$$D = D_a + K\Delta$$

where

D = true pavement deflection;
D_a = apparent pavement deflection;
Δ = vertical movement of front legs, i.e., twice difference between final and intermediate dial readings; and
K = 2.91 for U.S. Bureau of Public Roads type Benkelman beam.

Appendix B

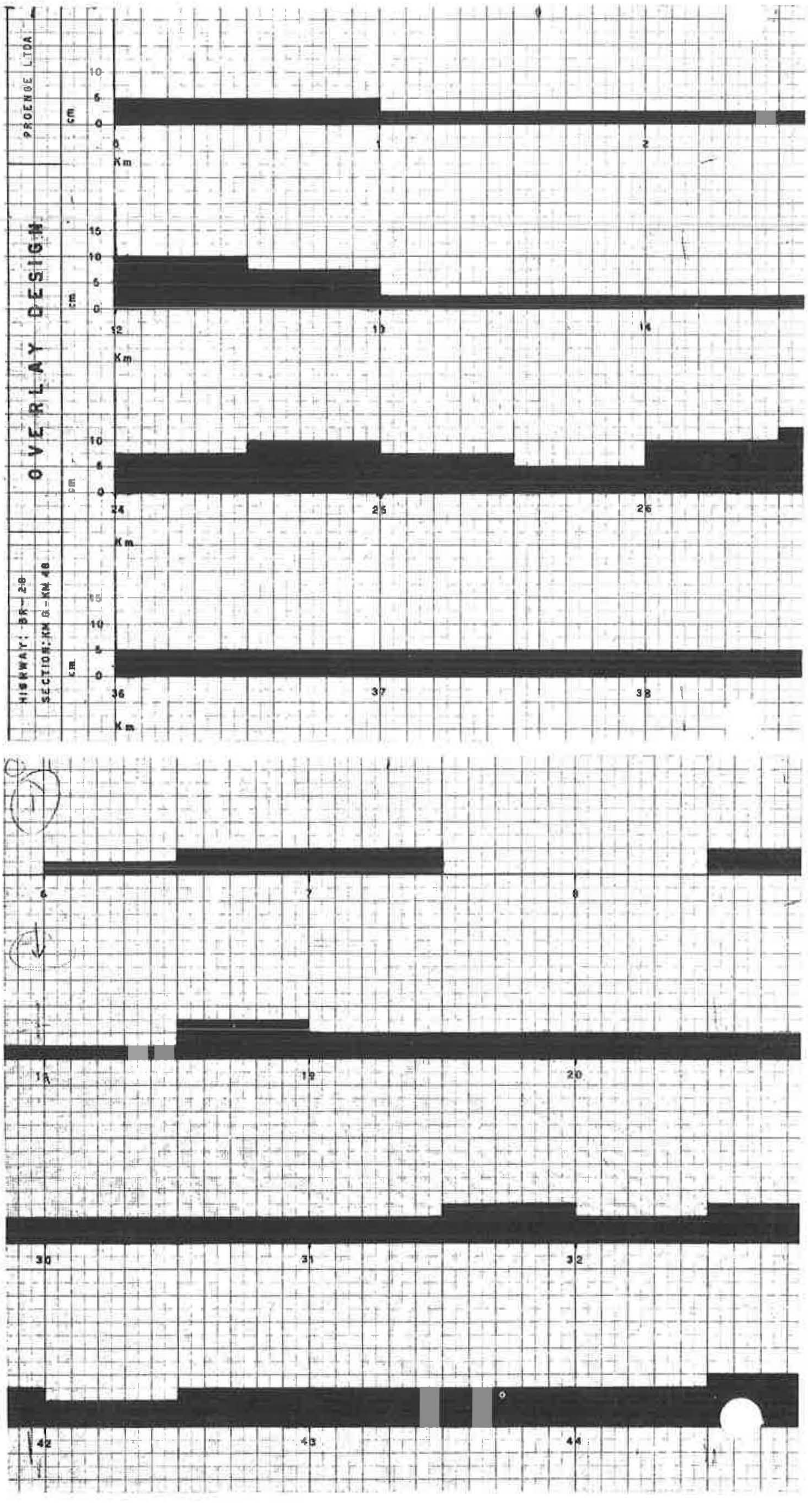
STATISTICAL STUDY OF DEFLECTION MEASUREMENTS AND OVERLAY THICKNESS DESIGN

(Highway: BR-28, Section: Km 0 - Km 48)

Sections Km - Km	Outer Wheelpath				Inner Wheelpath				Overlay Thickness Design	
	$\bar{D}$	σ	D_c	T (cm)	$\bar{D}$	σ	D_c	T (cm)	cm	in.
0- 0.5	43	16	75	5	49	14	77	6	5.0	2
0.5- 1.0	41	10	61	3	45	14	73	5	5.0	2
1.0- 1.5	36	13	62	3	37	11	59	2	2.5	1
1.5- 2.0	35	12	59	2	39	8	55	1	2.5	1
2.0- 2.5	35	12	59	2	39	12	63	3	2.5	1
2.5- 3.0	35	12	59	2	41	11	63	3	2.5	1
3.0- 3.5	34	17	68	4	39	13	65	3	5.0	2
3.5- 4.0	26	11	48	0	25	7	39	0	2.5	1
4.0- 4.5	41	13	67	4	39	15	69	4	5.0	2
4.5- 5.0	22	9	40	0	17	7	31	0	0.0	0
5.0- 5.5	19	9	37	0	17	6	29	0	0.0	0
5.5- 6.0	21	9	39	0	16	7	30	0	0.0	0
6.0- 6.5	35	15	65	3	32	9	50	0	2.5	1
6.5- 7.0	45	14	73	5	42	6	54	1	5.0	2
7.0- 7.5	41	15	71	5	33	11	55	1	5.0	2
7.5- 8.0	25	11	47	0	20	7	34	0	0.0	0
8.0- 8.5	19	8	35	0	17	7	31	0	0.0	0
8.5- 9.0	35	17	69	4	24	13	50	0	5.0	2
9.0- 9.5	61	15	91	7	50	15	80	6	7.5	3
9.5-10.0	61	15	91	7	51	16	83	6	7.5	3
10.0-10.5	59	23	105	9	52	20	92	8	10.0	4
10.5-11.0	59	22	103	9	56	20	96	8	10.0	4
11.0-11.5	54	14	82	6	49	15	79	6	5.0	2
11.5-12.0	58	20	98	8	58	19	96	8	7.5	3
12.0-12.5	65	19	103	9	63	15	93	8	10.0	4
12.5-13.0	59	17	93	8	52	14	80	6	7.5	3
13.0-13.5	43	11	65	3	41	11	63	3	2.5	1
13.5-14.0	35	8	51	1	34	10	54	1	2.5	1
14.0-14.5	30	11	52	1	35	9	53	1	2.5	1
14.5-15.0	32	8	48	0	36	10	56	2	2.5	1
15.0-15.5	38	9	56	2	41	9	59	2	2.5	1
15.5-16.0	38	6	50	0	35	8	51	1	2.5	1
16.0-16.5	35	8	51	1	35	7	49	0	2.5	1
16.5-17.0	39	8	55	1	38	8	54	1	2.5	1
17.0-17.5	37	10	57	2	37	8	53	1	2.5	1
17.5-18.0	39	13	65	3	39	12	63	3	2.5	1
18.0-18.5	36	9	54	1	29	10	49	0	2.5	1
18.5-19.0	50	17	84	7	43	15	73	5	7.5	3
19.0-19.5	48	8	64	3	44	12	68	4	5.0	2
19.5-20.0	38	10	58	2	37	13	63	3	5.0	2
20.0-20.5	37	12	61	3	35	7	49	0	5.0	2
20.5-21.0	52	8	68	4	49	7	63	3	5.0	2
21.0-21.5	39	11	61	3	39	6	51	1	5.0	2
21.5-22.0	52	14	80	6	51	14	79	6	5.0	2
22.0-22.5	81	13	107	9	81	15	111	10	10.0	4
22.5-23.0	95	20	135	12	97	23	143	13	12.5	5
23.0-23.5	81	20	121	11	74	19	112	10	10.0	4
23.5-24.0	67	17	101	9	63	18	99	9	10.0	4

STATISTICAL STUDY OF DEFLECTION MEASUREMENTS AND OVERLAY THICKNESS DESIGN (Continued)
(Highway: BR-28, Section: Km 0 - Km 48)

Sections Km - Km	Outer Wheelpath				Inner Wheelpath				Overlay Thickness Design	
	$\bar{D}$	σ	D_c	T (cm)	$\bar{D}$	σ	D_c	T (cm)	cm	in.
24.0-24.5	69	13	95	8	64	13	90	7	7.5	3
24.5-25.0	73	21	115	10	70	22	114	10	10.0	4
25.0-25.5	63	11	86	7	59	12	83	6	7.5	3
25.5-26.0	51	8	67	4	46	10	66	4	5.0	2
26.0-26.5	68	23	114	10	68	23	114	10	10.0	4
26.5-27.0	90	25	140	13	81	17	115	10	12.5	5
27.0-27.5	63	19	101	9	52	9	70	4	10.0	4
27.5-28.0	72	8	88	7	65	12	89	7	7.5	3
28.0-28.5	64	15	94	8	62	20	102	9	10.0	4
28.5-29.0	47	10	67	4	40	9	58	2	5.0	2
29.0-29.5	55	8	71	5	47	8	63	3	5.0	2
29.5-30.0	52	8	68	4	46	9	64	3	5.0	2
30.0-30.5	46	9	64	3	39	6	51	1	5.0	2
30.5-31.0	47	9	65	3	41	7	55	1	5.0	2
31.0-31.5	57	11	79	6	47	13	73	5	5.0	2
31.5-32.0	68	9	86	7	58	8	74	5	7.5	3
32.0-32.5	69	6	81	6	62	7	76	5	5.0	2
32.5-33.0	68	12	92	8	65	10	85	7	7.5	3
33.0-33.5	70	8	86	7	64	6	76	5	7.5	3
33.5-34.0	64	10	84	7	59	7	73	5	7.5	3
34.0-34.5	57	10	77	6	51	11	73	5	5.0	2
34.5-35.0	62	11	84	7	59	14	87	7	7.5	3
35.0-35.5	63	12	87	7	58	11	80	6	7.5	3
35.5-36.0	61	8	77	6	57	8	73	5	5.0	2
36.0-36.5	64	9	82	6	63	6	75	5	5.0	2
36.5-37.0	48	8	64	3	48	7	62	3	5.0	2
37.0-37.5	55	8	71	5	57	5	67	4	5.0	2
37.5-38.0	58	11	80	6	60	9	78	6	5.0	2
38.0-38.5	62	6	74	5	58	8	74	5	5.0	2
38.5-39.0	59	6	71	5	59	9	77	6	5.0	2
39.0-39.5	71	9	89	7	71	8	87	7	7.5	3
39.5-40.0	67	9	85	7	60	9	78	6	7.5	3
40.0-40.5	61	10	81	6	56	14	84	7	7.5	3
40.5-41.0	60	10	80	6	60	10	80	6	5.0	2
41.0-41.5	69	10	89	7	69	9	87	7	7.5	3
41.5-42.0	68	15	98	8	70	7	84	7	7.5	3
42.0-42.5	64	8	80	6	65	6	77	6	5.0	2
42.5-43.0	71	9	89	7	69	6	81	6	7.5	3
43.0-43.5	67	11	89	7	62	11	84	7	7.5	3
43.5-44.0	73	10	93	8	73	12	97	8	7.5	3
44.0-44.5	68	9	86	7	62	10	82	6	7.5	3
44.5-45.0	78	15	108	10	74	11	96	8	10.0	4
45.0-45.5	68	9	86	7	62	6	74	5	7.5	3
45.5-46.0	63	6	75	5	60	8	76	5	5.0	2
46.0-46.5	72	9	90	7	67	10	87	7	7.5	3
46.5-47.0	69	6	81	6	63	8	79	6	5.0	2
47.0-47.5	87	14	115	10	84	15	114	10	10.0	4
47.5-48.0	90	11	112	10	83	16	115	10	10.0	4



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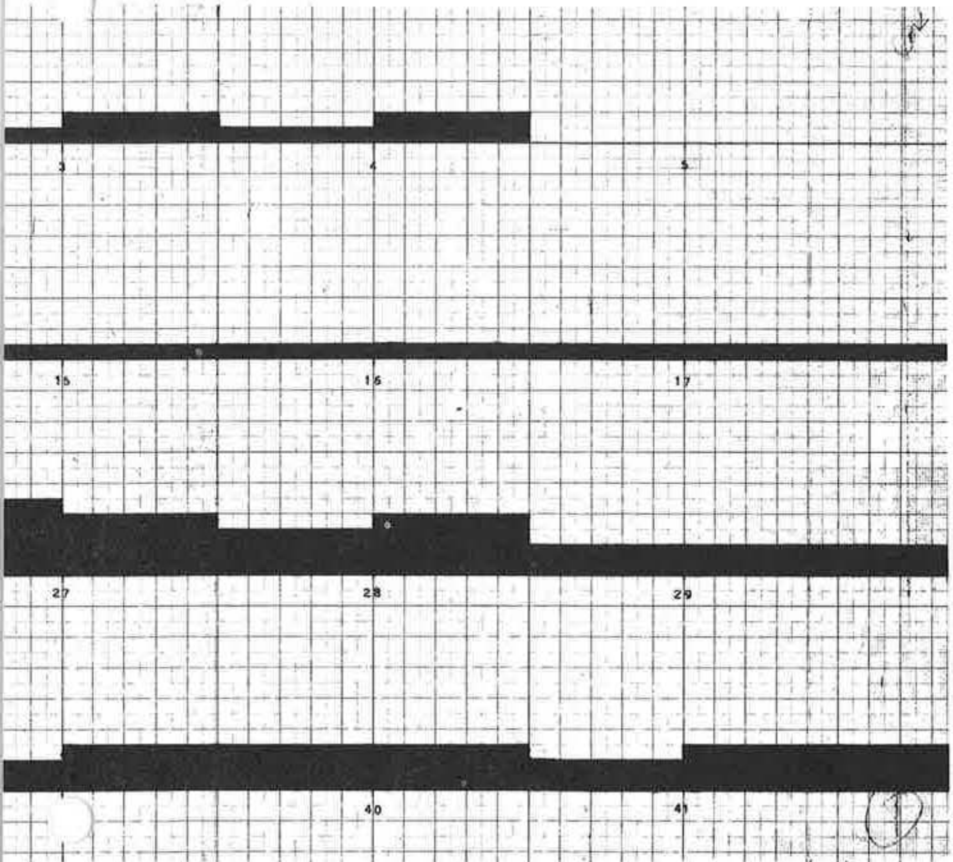

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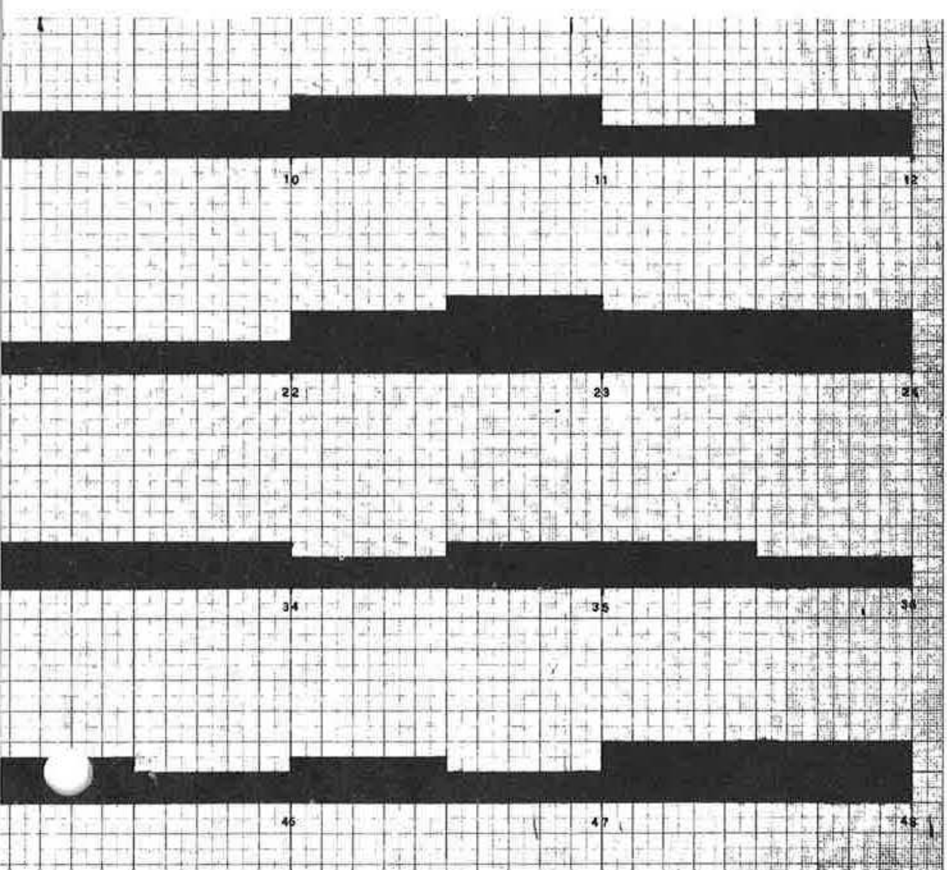

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Flexible Pavement Maintenance Requirements as Determined by Deflection Measurement

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This paper discusses the results of the use of the deflection method by the California Division of Highways for the evaluation of existing flexible pavements and the recommendation of suitable reconstruction. Since 1960, some 80 projects including state highways, county roads, and city streets, have been subject to deflection investigation by the Materials and Research Department of the California Division of Highways. The primary purpose of these investigations was the recommendation of appropriate corrective treatment. As a result of this intensive program, a large volume of data on the deflection attenuation properties of various roadway materials has been accumulated and is presented in this report, along with the results of individual deflection studies. The test procedure, method of evaluation of deflection data, and design criteria which have evolved are examined in detail. In addition, economical and practical factors involved in making a specific recommendation are discussed. A separate section of the report is devoted to a review of current deflection research, including work now being done on the establishment of maximum deflection criteria which may be adjusted for variations in traffic volume. A brief analysis of radius of curvature data obtained with the Dehlen curvature meter is also included.

•THE CALIFORNIA Division of Highways has used deflection measurement for the evaluation of flexible pavements since 1938. Until 1954, deflection measurements were obtained using General Electric travel gages and a later modification, the linear variable differential transformer gage. During these early years, the limited amount of deflection data available was used to evaluate flexible pavement sections subject to distress investigation. In 1951 a comprehensive deflection research program was initiated by the Materials and Research Department. The principal objective of this study was the establishment of a relationship between the level of pavement deflection and pavement performance or conditions. Secondary objectives included: (a) establishment of the relationship between single-axle load and pavement deflection, (b) determination of the effect on pavement deflection of wheel configuration, and (c) an examination of the relationship between pavement deflection and pavement temperature. Approximately 400 General Electric gage units were installed on 43 projects throughout California.

The test roadways included a wide variety of pavement structural sections because thickness of asphalt-concrete surfacing was a prime variable. Installations were made on both distressed and undistressed sections of the test roads. The rear axle loading for this and all subsequent deflection testing was 15,000 lb. The results and conclusions of this study were published in 1955 (1).

Paper sponsored by Committee on Maintenance of Bituminous Pavements and presented at the 45th Annual Meeting.

TABLE 1

MAXIMUM TOLERABLE DEFLECTION LEVELS

Pavement Thickness (in.)	Pavement Type	Max. Deflect. (in.)[a]
8	Portland cement concrete	0.012
6	Cement-treated base[b]	0.012
4	Asphalt concrete	0.017
3	Plant mix on gravel base	0.020
2	Plant mix on gravel base	0.025
1	Road mix on gravel base	0.036
½	Surface treatment	0.050

[a]For design purposes.
[b]Surfaced with bituminous pavement.

Figure 1. Traveling deflectometer.

Examination of the data from this study with respect to level of pavement deflection vs pavement condition permitted the establishment of tolerable deflection criteria for a variety of structural sections. The selected roads were, without exception, "main-line," carrying approximately 10 million or more equivalent 5,000-lb wheel loads (EWL) during their 10-yr design life. The criteria developed as a result of this study (Table 1) are of fundamental importance because they provided the basis for the practical application of pavement deflection data for the determination of the maintenance requirements of a distressed roadway.

These values are applicable primarily to California highways as the methods of mix design, seasonal weather variations, and the characteristics of the construction materials, notably asphalt binder, are peculiar to that state. They are somewhat conservative in comparison to the criteria established by other agencies.

The installation of linear variable differential transformer gages for deflection measurement was a tedious and time-consuming process. Because of this and the relatively high costs involved, only limited coverage was possible.

During the operational phase of the WASHO Road Test (1952-1954), A. C. Benkelman of the U. S. Bureau of Public Roads developed an instrument for measuring pavement deflection with the important advantages of versatility, simplicity, and speed.

With this device, upwards of 300 measurements per normal working day are possible. The development of the Benkelman beam, therefore, greatly augmented pavement deflection research and the use of pavement deflection measurements for overlay design.

Between 1955 and 1960, we developed a semiautomatic deflection device, based on the Benkelman beam principle, known as the traveling deflectometer (Fig. 1). This instrument combines a truck-trailer unit with dual probes for simultaneous deflection measurements under both sets of dual wheels. The device is electro-mechanical and is capable of uniform and consistent measurement of pavement deflections at 12½-ft intervals while traveling steadily at ½ mph. Between 1,500 and 2,000 individual deflection measurements are possible during the average working day. The development of the traveling deflectometer and the results of several of these early deflection studies were described in detail in 1962 at the International Conference on Structural Design of Asphalt Pavements (2).

By 1960, sufficient information on the deflection reduction properties of various roadway materials had been accumulated to permit reasonable estimation of the effectiveness of specific types of reconstruction for roadways evaluated by deflection study. The traveling deflectometer provided the means of obtaining a large volume of deflection test data within a relatively short period of time. Pavement deflection measurement for determination of roadway maintenance requirements, such as overlays, has been used with ever-increasing frequency since that time.

This report describes the evolution of a deflection test method by the California Division of Highways and presents the results of follow-up measurements on projects built in accordance with recommendations resulting from operational deflection studies. Detailed descriptions of five projects of particular interest are included. A portion of the report is devoted to a review of the scope and objectives of the current pavement deflection research program.

DEFLECTION TEST PROCEDURE

Prediction of Deflection Attenuation

Accumulation of deflection attenuation data was accomplished by two methods, the first of which was follow-up measurements over projects constructed according to recommendations resulting from deflection studies. Another very important source was test data from projects selected specifically for peculiarities in structural section, i.e., an unusually thick surfacing or base. From May 1960 to July 1965, some 80 separate deflection studies of an operational nature were conducted for the Division of Highways, counties, and cities involving deflection measurements and recommendations for corrective treatment for 250 individual roadways. As a result of this experience, the Division continues to accumulate a considerable amount of data on deflection attenuation. Figure 2 shows the sum total of experience to date with 17 completed projects. Percent reduction in deflection is plotted against increase in inches of gravel equivaence (the thickness of gravel necessary to produce a load-distributing and soil-restraining effect equal to that produced by the slab action of the thickness of the material being considered—refer to California Test Method No. 301-B). This plot is the basic tool for planning reconstruction of roadways based on deflection measurement. It not only establishes a general trend in the deflection reduction afforded by various thicknesses of base and surfacing, but also indicates the results of specific types of reconstruction on individual projects.

In addition to the general deflection attenuation trends resulting from this program, experience with the deflection method so far has shown the following.

1. In absolute terms, the reduction in deflection afforded by a given thickness of material is to a large extent dependent on the initial deflection level. In other words, the reduction in absolute units of deflection resulting from the placement from an AC layer is substantially greater at high deflection levels than at low deflection levels even though the percentage reduction might be the same in each case. Therefore, it is more realistic to estimate reduction in deflection in terms of percent of initial deflection rather than in terms of 0.001 in. per inch of resurfacing.

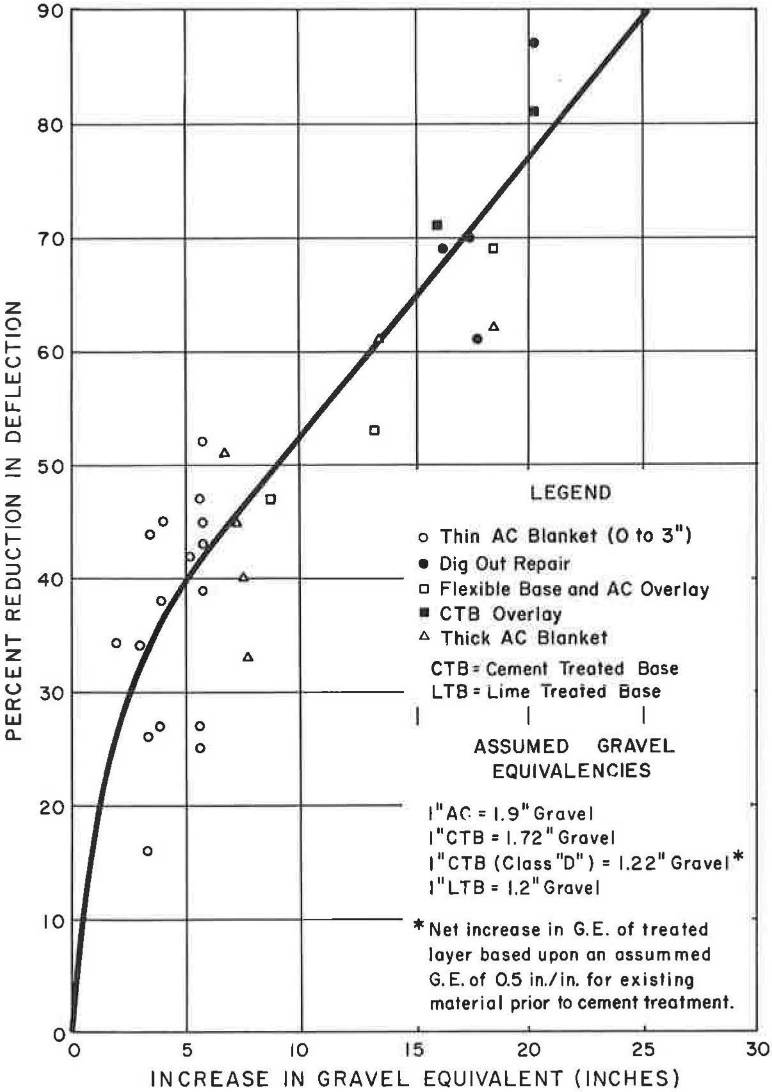

Figure 2. Reduction in deflection resulting from pavement reconstruction.

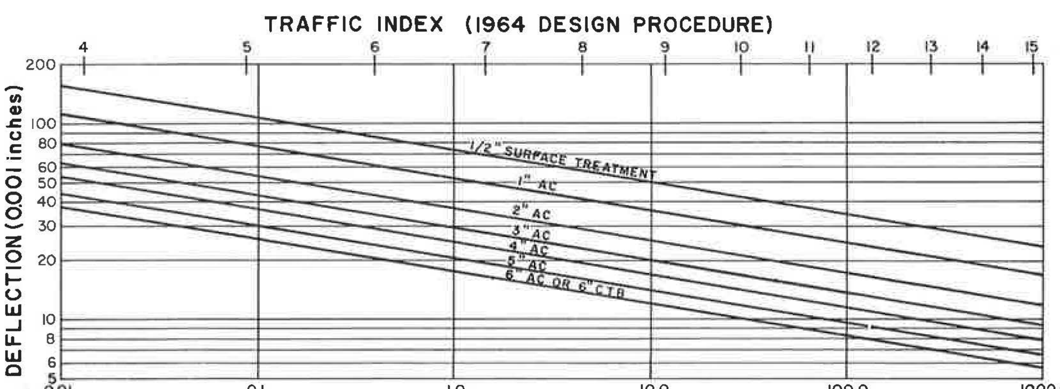

Figure 3. Variation in tolerable deflection, based on AC fatigue tests.

2. A significant reduction in deflection usually occurs during the first year of operation, presumably due to the additional curing of AC surfacing and traffic compaction.

3. The most economical reconstruction involves, insofar as possible, complete utilization of an existing structural section even though the surfacing may be badly cracked and spalled.

4. The highest rate of deflection reduction occurs with relatively thin treatments. This rate of attenuation tends to diminish with an increase in gravel equivalence.

5. The reduction in deflection resulting from the cement treatment of an in-place material is somewhat greater than indicated by existing gravel equivalent factors in California.

Variation in Tolerable Deflection with Traffic

The present limiting deflection criteria were established on the basis of data from heavily trafficked test roadways. It has been long recognized that application of these criteria to secondary state highways, county roads, and city streets would be unrealistic and uneconomical. For this reason, we have developed an interim method for adjustment of tolerable deflection level according to variations in traffic volume. This adjustment is based on AC surfacing fatigue tests made some time ago, which indicated that although the fatigue life of individual AC specimens varied widely (presumably due to variation of mix design, age, and number of previous traffic loadings) the slopes of their load repetition vs deflection lines were relatively uniform when plotted as logarithmic functions. By using an average AC surfacing fatigue line slope and pivoting lines through known deflection criteria at the 9.0 TI (traffic index) level, Figure 3 was developed to make rule-of-thumb adjustments in tolerable deflection for varying traffic volumes. (Traffic index is an exponential function of total EWL anticipated on the highway between the time construction is completed and the end of the design period—Calif. Test Method No. 301-B.) Although these curves are based solely on laboratory surfacing fatigue data and have not yet been correlated with field performance, they appear reasonable within the ranges of 6.0 to 10.0 TI.

Selection of Test Section

Before making deflection measurements on a certain road, the project file is studied for information on variations in structural section, traffic volume, foundation and drainage conditions, and unusual occurrences during construction which may have affected the performance of the roadway. From this and visual examination, test sections considered representative are selected. Approximately 1,000 ft per centerline mi are tested on each project. Deflection test data are separated into categories of fill, cut, cracked, uncracked, travel lane, passing lane, and inner and outer wheel path (OWP). Further breakdowns or divisions are established as warranted by peculiarities of the project. Examination of average deflections for each category can frequently indicate the nature or cause of early pavement distress and the practicability of utilizing more than one type of corrective treatment. In cases where deflection is relatively uniform, an evaluated (deflection value at which 80 percent of the measurements are lower and 20 percent are higher) deflection level (80 percentile) is established by recombining all OWP readings from the test section. This value reflects the deflection characteristics of the section as a whole rather than isolating possible causes of distress or placing undue emphasis on an isolated condition.

Selection of Required Maintenance Treatment

The problem of recommending suitable reconstruction is not simply a matter of establishing a representative deflection level and prescribing a treatment which would reduce this deflection to a tolerable limit. Several other factors are considered to arrive at a satisfactory design; these are (a) existing vertical controls (curbs and gutters); (b) anticipated use of the roadway; (c) extent and nature of cracking; and (d) anticipated traffic volume.

TABLE 2

DEFLECTION DATA FROM TYPICAL CITY STREET IN CALIFORNIA

Test Section	Deflection (in.)			Appearance
	Mean		Evaluated	
	OWP	IWP	(80% level)	
1	0.055	0.028	0.106[a]	Intermittent alligator cracking and shrinkage cracking.
2	0.043	0.031	0.064[b]	Continuous to intermittent alligator cracking in both wheel paths of all lanes.

[a]Based on 35 individual deflection measurements.
[b]Based on 28 individual deflection measurements.

The existence of curbs and gutters or the presence of an excellent passing lane next to a distressed travel lane often makes the use of a travel lane digout feasible. Where no such vertical control exists and a major reconstruction is warranted, a flexible base or cement-treated base (CTB) with an AC blanket is usually recommended so that the residual strength of the old pavement can be incorporated into the new construction.

The anticipated future use of a roadway frequently determines whether we shall "live" with a deflection condition through utilization of a thin blanket or eliminate the problem with major reconstruction.

The extent and nature of cracking is frequently very important in determining whether a blanket will act independently of the old surfacing or become an integral part of the existing surfacing, thereby increasing surface rigidity with a corresponding decrease in the level of tolerable deflection.

The presence of large block or ladder-type cracks indicates that the existing surfacing has a good deal of residual slab strength and could thus be expected to act in conjunction with a new blanket. Thus, the AC surfacing would consist of the original and the repair blanket, acting as a unit. Because of this, the tolerable deflection level would be much lower than that for a new blanket applied to a continuously cracked AC surfacing in which surface distress is in the form of relatively small blocks as is sometimes the case with badly alligator-cracked roads. Here, because the new blanket can be considered independent of the old, the tolerable deflection level can be assumed to be determined by the thickness of the new blanket only.

The deflection method for the design of reconstruction is assumed to be valid when roadway distress is attributable to excessive compression and rebound to the structural section. Evidence of the instability of the structural section as manifested by permanent path rutting or indication of significant permanent deformation on the deflection traces reveals a problem beyond the scope of the deflection method. In these cases, design of corrective treatment is based on the standard California R-value procedure.

To illustrate the method of analysis and procedure for recommendation of corrective treatment based on deflection data, a typical case history of a particular roadway will be examined. The information in Table 2 was acquired during a recent deflection investigation of the streets of a medium-size city in the central valley of California. The roadway had a structural section consisting of 2 in. of AC surfacing over 4 in. of aggregate base over 4 in. of aggregate subbase. The design TI was assumed to be 6.5.

The evaluated deflection levels for the two test sections ranged from 0.064 to 0.106 in. Test section No. 1, however, had a mean OWP deflection level of 0.055 in. The wide discrepancy between the mean level, 0.055 in., and the evaluated (80 percentile) level, 0.106 in., indicates that the evaluated deflection level was greatly influenced by a few isolated high readings and, thus, is not representative of the test section as a whole. With this in mind, the evaluated deflection level of test section No. 2 (0.064 in.) is selected as the design deflection level. Based on a TI of 6.5 and 3-in. AC surfacing, it is determined from Figure 3 that a deflection level of 0.030 in. can be tolerated. It is, therefore, necessary to effect a reduction in the deflection level of 0.064 in. minus 0.030 in., or 0.034 in. This requires a 0.034 in./0.064 in. = 53 percent reduction in deflection. Figure 2 shows that an increase of 10.5 in. in gravel equivalence is required to reduce the deflection level by 53 percent. For a 3-in. AC surfacing the gravel equivalence is 3.0 × 1.9 in. = 5.7 in. It will, therefore, be necessary to provide 10.5 in. - 5.7 in. = 4.8 in. of additional gravel. A possible reconstruction would, therefore, be the placement of a 3-in. AC surfacing over 5.0 in. of aggregate base directly over the existing roadway.

Another practical approach to the same problem which would cost less takes into consideration the type of distress on the roadway. Here intermittent to continuous alligator cracking occurs in both wheel paths. Because alligator cracks are usually small (2 to 5 in. in diameter) it can be reasonably assumed that the existing pavement will act independently of the new surfacing in much the same manner as an aggregate base. Therefore, consideration should be given to the possibility of placing a thin AC blanket which would permit a higher tolerable deflection level. This approach could be considered "living" with a high deflection condition rather than eliminating it by a major reconstruction. For a 2-in. AC surfacing, Figure 3 shows a tolerable deflection of 0.040 in. It would, therefore, be necessary to reduce the design deflection level of 0.064 to 0.040 in. which requires a 38 percent reduction in deflection. From Figure 2, a 2-in. AC blanket (3.8-in. gravel equivalence) provides a 37 percent reduction in deflection. This is considered close enough to recommend a 2-in. AC surfacing for repair.

In either case, isolated areas of high transient deflection or advanced distress should be subject to substantial digout type repair before the application of the corrective treatments.

RESULTS OF SPECIFIC INVESTIGATIONS

The following are brief histories of five past deflection studies which illustrate unusual problems and conditions. The criteria used in recommending corrective treatment for these projects have been changed somewhat due to a recent revision of gravel equivalences of base and AC surfacing and modification of the deflection attenuation curves which were used at that time.

V-Mon-118-Salinas

In July 1961 District V materials personnel sampled the in-place structural section of this facility at several locations. Within the city of Salinas the asphalt surfacing varied from 3 to 6 in. in thickness and the base material varied from 2½ to 11 in. Average passing and travel lane deflection measurements taken in August 1961 are given in Table 3. Based on the average deflection levels of 0.067 and 0.058 in. for the travel lanes and the deflection design criteria in use at that time, it was determined that an increase in gravel equivalence of 12 in. was required. For the passing lanes, because of their generally good appearance, lower deflection levels and lighter traffic load, an increase of only 4 in. in gravel equivalence was recommended. The existence of curbs, gutters, and buried utility lines near the surface placed severe limitations on the thickness of both an overlay or digout type repair for the travel lanes. As a result, the travel lanes were scarified to a depth of 8 in. On removal of the existing base and surfacing, 8 in. of an untreated Class 2 aggregate base was placed and compacted, bringing the roadway back to original finished grade. Both the passing and the travel lanes were then blanketed with 3 in. of AC surfacing.

The net result of the reconstruction of the travel lanes was the replacement of a cracked AC surfacing with an uncracked 3-in. AC surfacing and the replacement of the existing base with a new lift of aggregate base material. The placement of a 3-in.

TABLE 3

DEFLECTION DATA FROM PROJECT V-MON-118-SALINAS

Location	Lane	Mean OWP Deflect. (in.)
Between Lincoln and Vale St.	Westbound travel	0.067
Between Stone and Capitol St.	Westbound passing	0.034
Between New St. and West City Limits	Westbound travel	0.038
Between Capitol and Stone St.	Eastbound travel	0.058
Between Riker and Capitol St.	Eastbound passing	0.042
Between Clark and New St.	Eastbound travel	0.029

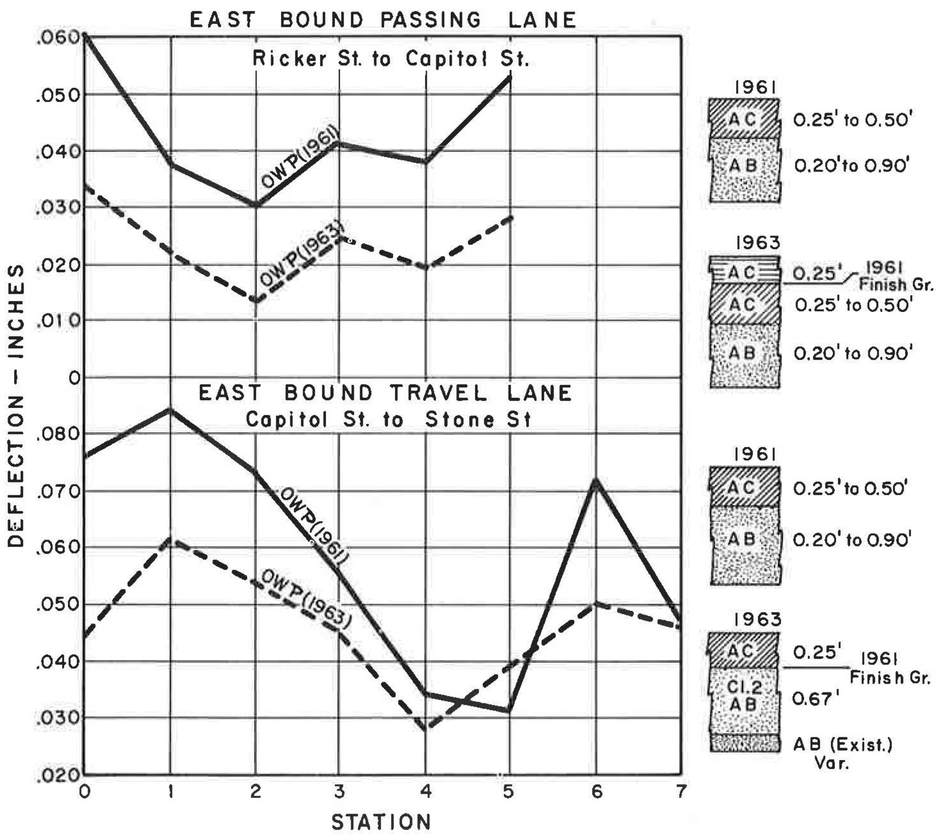

Figure 4. Deflection measurements on test pavement before and after reconstruction.

TABLE 4

PROPERTIES OF ASPHALT BINDER RECOVERED
FROM CORES TAKEN ON PROJECT V-SLO-2-PBch, E

Station	Binder	Core	Depth (in.)	Properties	
				Pen., 77F	Duct., 77F (5 cm/min)
"N" 244+64[a]	120-150 Pen.	1954 Surf. course	0-1½	7	8
	200-300 Pen.	1949 Surf. course	3-¾-8	33	100+
"N" 240+50[a]	120-150 Pen.	1954 Surf. course	0-1½	3	0
	200-300 Pen.	1949 Surf. course	3-¾	13	22

[a]Northbound travel lane, OWP.

Figure 5. Road V-SLO-2-PBch, E, northbound travel lane.

contact blanket over the passing lanes, however, resulted in the full utilization of the residual strength of the old surfacing. The results of deflection measurements before and after reconstruction for one test section are shown in Figure 4. The average percent reduction in mean OWP deflection was significantly greater in the passing lane section than in the travel lane digout sections (45% as opposed to 16%). This project illustrates a truism with regard to overlay design, i.e., whenever possible, reconstruction should fully utilize the residual structural strength of an existing roadway which more often than not is considerable even for badly cracked pavements.

V-SLO-2-PBch, E

Originally constructed in 1949, the structural section included 4 in. of AC surfacing over 6 in. of crusher run base covering 12 in. of imported subbase material. Because of the appearance of early surface distress, a portion of the roadway was resurfaced in 1954 with a 3-in. AC blanket. In 1959 District V materials personnel conducted an investigation to determine the cause of surfacing distress which had reappeared since the placement of the 1954 contact blanket. Even though cracking was almost continuous throughout the length of the project, the structural section was entirely adequate in thickness and quality. It was, therefore, suspected that excessive pavement deflection had induced premature fatigue cracking of the surfacing.

In January 1960, materials and research personnel made visual observations, pavement deflection measurements, and cored into the structural section at two locations. Visual inspection of the roadway revealed almost continuous alligator cracking with some spalling in both the IWP (inner wheelpath) and OWP of the outer lane (Fig. 5). Little evidence of rutting or pumping was observed. Deflection measurements were uniformly low, averaging 0.016 in. in the travel lane OWP and 0.012 in. in the passing lane OWP. The results of tests on AC surfacing cores, however, indicated that the asphalt binder in the 1954 surface course had reached a state of hardness such that it could not withstand even the relatively low deflections characteristic of the roadway. This is shown by the results of tests on recovered asphalt from cores taken at two different locations on the roadway (Table 4).

The 1954 surfacing binder reached a critical state of hardness as indicated by recovered penetrations of 7 and 3 and ductilities of 8 and 0. These values show a much greater degree of hardness than that found for the 1949 surface course with recovered penetrations of 33 and 13 and ductilities of 100+ and 22. As a result, the 1954 overlay surfacing was cracked to a much greater extent than even the original 1949 pavement.

In view of the low deflection characteristic of the travel lanes, it was recommended that a 2-in. AC blanket be placed over the entire roadway. Because the 1954 surface course had cracked into relatively small blocks, it was believed that the possibility of reflective cracking into the new blanket would be minimal. A 2-in. AC blanket was placed as recommended, in 1960. To date, after nearly 5½ yr of service, there has been no further manifestation of surface distress. The use of deflection measurements, therefore, resulted in a real savings since, based purely on visual observation, a much greater degree of reconstruction would normally have been recommended.

Greenwood Avenue, Sanger

This roadway is typical of the many county and city streets tested during the past two to three years over which surprisingly low levels of transient deflection were noted in spite of relatively thin structural sections. In this case, the structural section consisted of 2 to 4 in. of oiled earth and rock. Visual appearance of the roadway was generally good with isolated areas of shrinkage and alligator cracking. No wheelpath depressions or other evidence of instability were observed. Deflection measurements made in April 1965 produced relatively low evaluated deflection levels ranging from 0.023 to 0.038 in. which, based on existing criteria for a 3-in. AC surfacing at 6.0 TI (0.035 in.), did not indicate a need for corrective treatment. Consequently, a double screening seal coat was recommended to improve roadway appearance and seal off the section although, based on conventional strength tests, it is likely that a much heavier reconstruction would have been indicated. The good visual appearance and low deflection level of this facility can probably be attributed to age-hardening of the AC coupled with an increase in load-carrying capacity of the basement soil resulting from successively heavier applications of traffic throughout the years. It is unlikely, therefore, that a similar but newly constructed structural section would prove successful in view of the heavier volume of traffic on the facility immediately after construction.

V-SLO-2-B (Between Atascadero and Paso Robles)

This project was constructed to its present 4-lane divided alignment in 1951. The original structural section consisted of 4 in. of AC surfacing and a variable thickness of base material which had 2 to 3 percent cement added to the upper 8 in. In 1958, as a result of extensive block cracking in the travel lanes, a 1-in. AC blanket was placed over the entire roadway. This was in addition to regular maintenance of a sporadic nature which, by 1960, was estimated to cost nearly $2,000/mi/yr. In June 1961, just before a deflection study, a field review of the road was completed. Visual observations indicated that the travel lanes were badly cracked, with spalling evident in some areas. Only very slight distress was observed in the passing lanes. The nature of the cracking indicated reflection from block cracks in the cement-treated base as the primary cause of surface distress. Mean OWP deflection levels ranged from 0.032 to 0.051 in. and individual measurements in the travel lane varied from 0.012 to 0.084 in. These data confirmed the results of visual observations by indicating that the cement-treated base was providing little or no slab strength. The relatively high mean OWP deflection levels over the uncracked sections (0.018 and 0.025 in.) suggested that even these areas were in a state of incipient distress. This facility (US 101), one of the two major highways between the San Francisco Bay area and Los Angeles, is subject to extremely heavy truck traffic. Because of this and its relatively high level of transient deflection, a major repair was indicated. It was estimated that an AC blanket of sufficient thickness to reduce travel lane deflections to a tolerable level would have required substantial shoulder reconstruction and was not necessary in the passing lane. It was, therefore, recommended that the existing AC surface and cement-treated base be pulverized to a depth of 10 in. below the existing finish profile grade in the travel lanes only and that sufficient cement be added for the construction of new cement-treated base 8 in. in thickness having a minimum 7-day compressive strength of 500 psi. It was further recommended that the travel lane be blanketed with 2 in. of AC over the cement-treated base, returning it to its original grade and that both lanes then be surfaced with a 2-in. AC blanket. With two minor modifications, the roadway was reconstructed as recommended. The thickness of CTB was increased from 8 to 10 in. Also, a ¾-in. open-graded AC surfacing was placed over both lanes in addition to the dense-graded AC blanket originally recommended. The results of deflection measurements over a typical test section are shown in Figure 6. The level of deflection in the OWP was reduced by an average of 87 percent to below 0.005 in.

The results of this and similar projects demonstrate that successful cement treatment of existing base and surfacing materials can greatly strengthen an existing section without significantly raising profile grade. Hence, this technique has proved economical and effective for the reconstruction of roadways subject to the existing vertical control of curbs, gutters, or undistressed interior lanes.

VI-Kin, Tul-135-B, A

Where it is possible to utilize an existing structural section in its entirety, placement of reconstruction directly over existing surfacing permits comparable results with thinner reconstruction. This is demonstrated by the results of the deflection study on road VI-Kin, Tul-135-B, A, which at the time of the investigation had a structural section consisting of 3 in. AC, 6 in. of low strength (Class C, 1 to 2½ percent) CTB, 5 in. of aggregate base, and 11 in. of imported borrow. The results of deflection measurements before reconstruction are given in Table 5.

Based on an average deflection level for the cracked areas of 0.047 in. the design criteria in use at that time indicated a need for an increase in gravel equivalence of 15 in. It was recommended that this be accomplished by scarifying the existing surfacing and base to a depth of 8 in. to be followed with an addition of sufficient cement to form a CTB with a minimum compressive strength of 500 psi in 7 days.

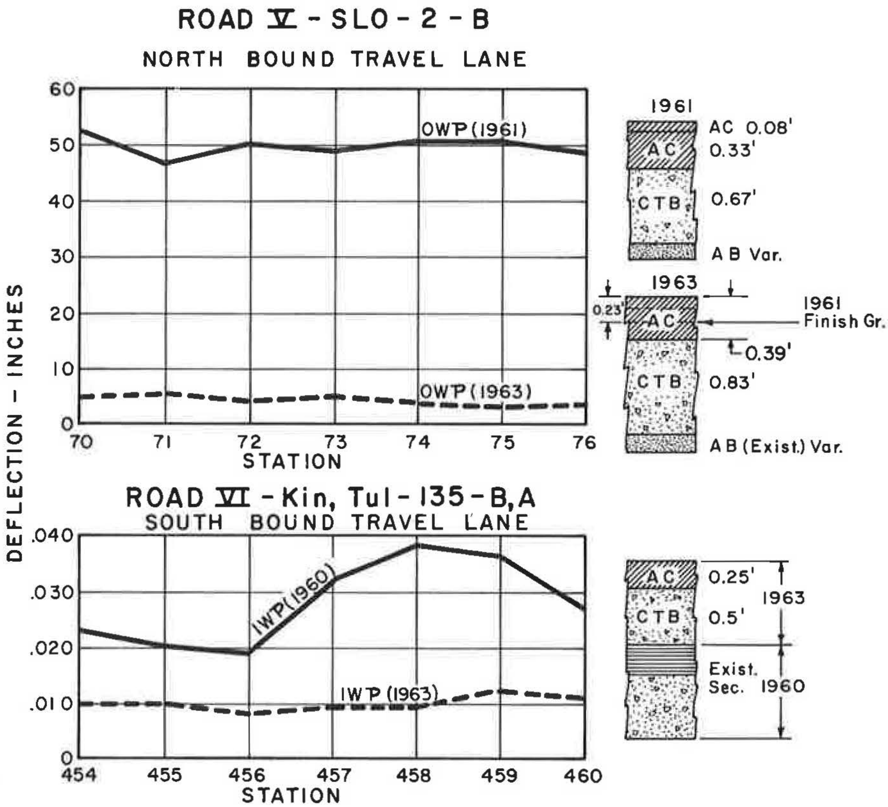

Figure 6. Deflection measurements over test section.

TABLE 5

DEFLECTION DATA FROM PROJECT VI-Kin, Tul-135-B, A

Condition	Mean Deflection (in.)	Evaluated Deflection (in.)
(a) Northbound Lanes		
Uncracked		
OWP	0.036	0.049
IWP	0.028	0.030
Cracked		
OWP	0.057	0.084
IWP	0.047	0.066
(b) Southbound Lanes		
Uncracked		
OWP	0.034	0.042
IWP	0.032	0.038
Cracked		
OWP	0.049	0.068
IWP	0.031	0.038
(c) Summary		
Uncracked	0.032	0.042
Cracked	0.047	0.068

It was further recommended that the entire roadway was to be blanketed with a 3-in. AC surfacing. Because of the absence of vertical controls, the district elected to place a 6-in. layer of cement-treated base and a 3-in. AC blanket directly over the original roadway, which provided for an increase in gravel equivalence of 16 in. Figure 6 shows deflection data before and after reconstruction from one test section. Deflection measurements indicated a reduction in transient deflection level by an average of 71 percent. This project was considered quite successful since the deflection levels after application of corrective treatment were reduced below the critical level.

Full utilization of the deflection test method is of such recent origin that only a portion of the projects subject to deflection study and corrective treatment have been constructed. Even so, the potential of the deflection test method for effecting substantial savings in the maintenance and reconstruction of existing roadways has been convincingly demonstrated on several occasions. Donald Winton (4) stated that large savings were realized by following the recommendations resulting from a pavement deflection study of the Fresno city streets. The Materials and Research Department recommendations, when compared to the cost of the previously anticipated reconstruction, allowed a cost reduction of several hundred thousand dollars.

The costs involved in making the typical deflection study have so far been quite reasonable in consideration of the coverage possible with the traveling deflectometer. Normally ten to twelve 1,000-ft road sections representing approximately 10 mi of roadway are tested during a given working day. The cost of the deflectometer crew and equipment is approximately $275 per day, not including flagmen who are usually supplied by the highway district, city, or country requesting the survey. Most deflection studies have cost between $500 and $1000, including the completed report.

CURRENT DEFLECTION RESEARCH

Pavement deflection research in California is now concentrated in three general problem areas. The first and largest program involves the establishment of a tie be tween tolerable deflection levels, structural section, and traffic volume, or traffic index. As mentioned earlier, the present limiting criteria for maximum allowable deflection were established in 1955 as a result of a comprehensive study throughout the state. It is not unlikely that the values developed as a result of this investigation tend to be conservative when applied to roadways with light and medium traffic volumes, because the initial investigation was conducted over heavily trafficked roads (9.0 ± TI).

Another important reason why these values may be subject to some alteration is the improvement in asphalt-concrete durability and thus AC surfacing fatigue resistance, which has undoubtedly been brought about by a recent modification of our AC mix design method. The principal objective of this study, therefore, is the establishment of new maximum deflection criteria, which make allowances for a more durable asphalt-concrete and which can be adjusted for variations in predicted traffic volume. This project, in which the U. S. Bureau of Public Roads is cooperating, has been under way for over a year. Twenty-five roadways throughout the state, meeting the following requirements, were selected for a 5-yr comprehensive pretest program:

1. They are AC-surfaced roadways over which reliable traffic data are available.
2. They are newly constructed roadways which have not been in operation for more than 3 yr.
3. They have a reasonably large variation in structural section and deflection level.

The test program, which is being carried out during the spring of each year, consists of deflection measurements obtained with the traveling deflectometer over selected test sections of each roadway. These sections consist of three to five 1,000-ft lengths of the roadway, depending on the size and the nature of the project. In addition to deflection measurements, a precise crack survey and rut depth determination is made over each test section. AC cores 4 and 12 in. in diameter are taken in and between the wheelpaths. These samples are subject to flexural strength, microviscosity, permeability, stability, cohesion, and density tests. The yearly test program outlined previously will be continued until each test section manifests distress to a predetermined level considered to be failure. It is believed that this study is of sufficient

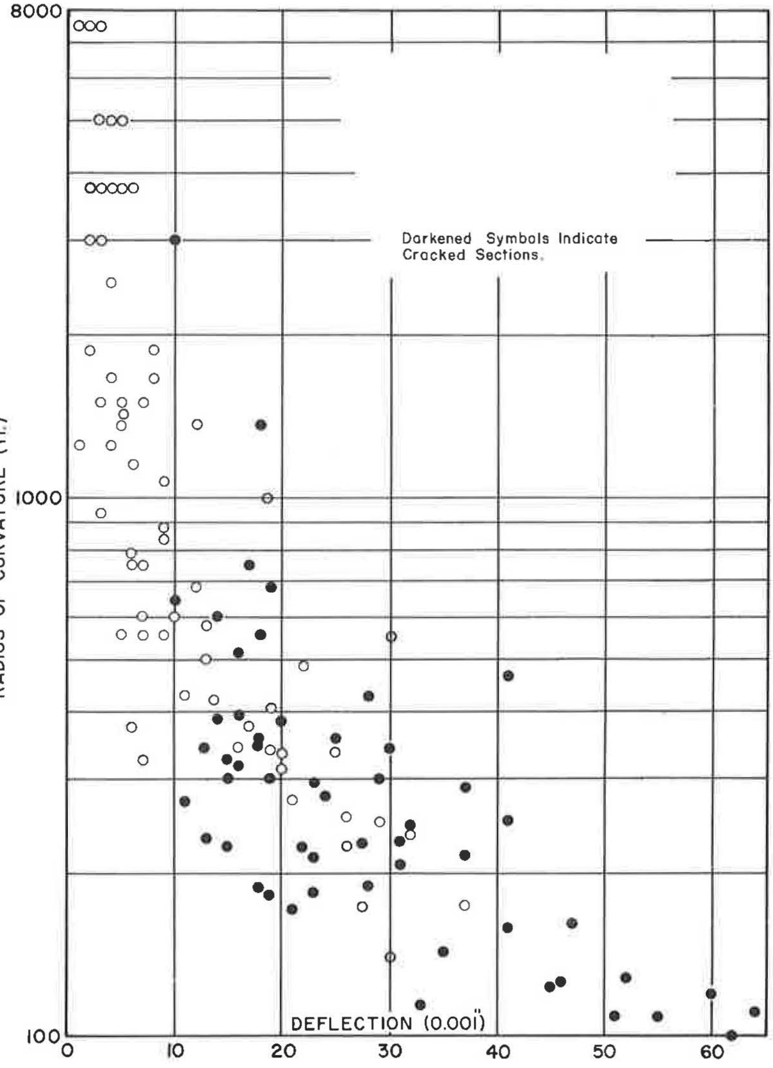

Figure 7. Radius of curvature (curvature meter) vs deflection cement-treated base.

scope to permit a valid appraisal of the effect of transient deflection, fatigue characteristics, asphalt quality, mix design, and traffic volume, on asphalt-concrete performance.

The second area of study involves the determination and analysis of area of influence or radius of curvature of a pavement under load, and the relationship to pavement performance. It would seem entirely reasonable, as many authorities contend, that pavement performance and condition are related more directly to severity of bending or area of influence than to lineal deflection measurement alone. Dehlen (3), proponent of the radius of curvature concept, presented a new device for measurement of radius of curvature, with data resulting from its use. This device, called a curvature meter, is an aluminum bar approximately 1 ft in length with an Ames dial and probe fixed in the center. By placing it between the wheels, it is possible to measure the middle ordinate of a curve 1 ft in length in the deflected basin from which a radius of curvature can be calculated.

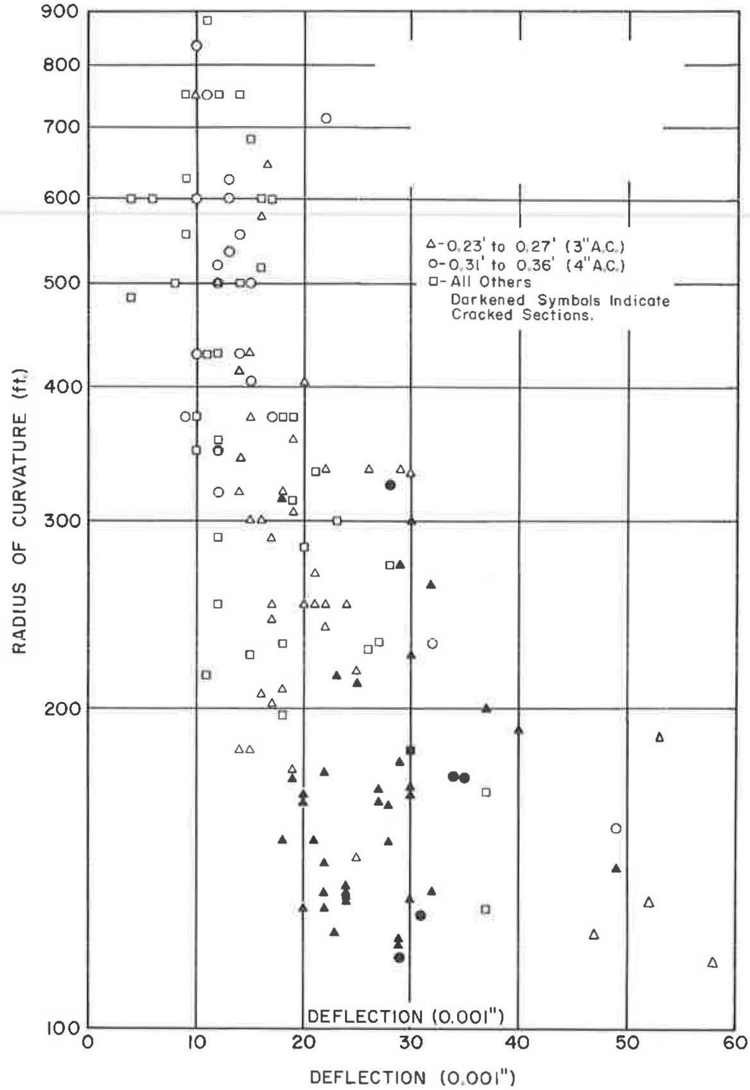

Figure 8. Radius of curvature (curvature meter) vs deflection aggregate base.

This device has been fabricated by us and used on several projects in conjunction with conventional deflection measurements (Figs. 7 and 8). In Figure 7, radius of curvature calculated from curvature meter measurements vs lineal deflection are plotted for cement-treated base construction. The open circles represent unfailed areas, with the closed dots representing cracked sections of the roadway from which the measurements were taken. Although relatively few data are available, it appears that lineal deflection was the best predictor of cement-treated base performance as there is a clear-cut demarcation between cracked and uncracked measurements at the 0.012 in. deflection level. For radius of curvature this demarcation is less clear-cut; however, a critical radius appears to be in the range of from 500 to 700 ft.

Figure 8 shows a similar plot for aggregate base structural sections. In this case, the radius of curvature appears to be the best forecaster of pavement performance, with a critical radius of curvature of approximately 200 ft. The critical zone for lineal deflection occurs at approximately 0.020 in., although there is a considerable overlapping between 0.020 and 0.030 in. Based on the limited amount of data in Figures 7 and 8, it would be difficult to determine whether lineal deflection or radius of curvature manifests a clear-cut superiority as an indicator of future pavement performance. Because of its simplicity and compactness, in addition to its sensitivity in a very critical zone of the deflected basin, further evaluations of the instrument will be made on projects subject to deflection study.

Attempts made to relate various functions of deflectometer trace shape to pavement condition have so far proved inconclusive. This is possibly because the zone of critical bending is confined to a very small portion of the trace, thus reducing sensitivity.

CONCLUSION

Significance of Pavement Deflection

With a steadily increasing amount of reconstruction of existing roadways, the need for a method to determine the minimum corrective treatment required to restore an existing roadbed to a state in which it may serve present-day traffic and provide maintenance-free service for an extended period has become increasingly important.

The problem encountered in the design of reconstruction is, of course, entirely different from that which occurs with all new construction. In the latter case, samples of basement or embankment soils are tested statically under moisture and density conditions estimated to be the worst that will occur during the lifetime of the pavement. From the results of these tests, subgrade bearing capacity is determined with which the necessary thickness of base or subbase can be calculated to provide the required cover in accordance with the appropriate design formula. The design of reconstruction for an existing roadway presents quite another problem, however, since the most economic reconstruction requires that full benefit be derived from the materials already existing in the structural section. In this case, a laboratory strength value cannot be considered quite valid, since the conditions of moisture and density assumed during preliminary design may not have occurred. Also, it is a well-known fact that many years of successively heavier traffic loadings tend gradually to increase in-place soil strength. Another factor which is difficult to evaluate is the residual strength of an asphalt-concrete surfacing or cement-treated base. Here, the hardening or curing induced by age may lend considerable slab strength to the system even though there is continuous visible distress. The real significance of pavement deflection data, therefore, is that it gives the highway engineer an indication of the total in-place structural strength of an existing roadway and, thus, provides an extremely valuable tool for the determination of the minimum degree of required reconstruction.

ACKNOWLEDGMENTS

The results of this statewide pavement deflection research program and the preparation of this report were accomplished under the general direction of John L. Beaton, Materials and Research Engineer of the California Divison of Highways. The authors wish to acknowledge the contributions of the many individuals who have participated directly in obtaining, tabulating, and analyzing the deflection test data used for this report. We are particularly grateful to Joseph Hannon and Harold Munday of this department.

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General Discussion

A. C. BENKELMAN, Consulting Engineer

•THESE PAPERS are concerned with pavement rehabilitation, a problem that is becoming increasingly important. It essentially concerns determination of the additional thickness of overlay material needed to restore the riding qualities of pavements that have deteriorated due to environmental effects or the additional thickness needed to increase the structural capacity of distressed pavements to accommodate existing traffic or an estimated growth in traffic. At one time this was done solely on the basis of judgment and experience of the engineer. These papers demonstrate that an increasing amount of interest is currently being shown in the use of deflection data to arrive at the answers needed. To achieve an appreciable level of success, one must know, within reason, the tolerable deflections for different pavements, and the inherent ability of many different materials to reduce deflection when used as overlay.

The paper by Zube and Forsyth contains considerable information on these two items. By converting the values on the horizontal scale in Figure 2 in the report (Increase in gravel equivalent thickness) to inches of asphalt concrete using the equivalence cited (1 in. AC = 1. 9 in. gravel), it was possible to compute the percent of deflection reduction for each inch of material. The values range from 29 percent per inch of the material per inch of thickness to 14 percent for 3 in. and 9 percent for 6 in. These values, as indicated in the text, show that the degree of reduction or attenuation in deflection depends on the thickness of the AC layer, i. e., as stated, "The highest rate of deflection reduction occurs with relatively thin treatment."

Similar values, developed in Carneiro's paper, indicate that the deflection reduction for 1 in. of a typical grade of asphalt-concrete would amount to about 17 percent per inch for a 1-in. thickness, 14 percent for 3 in., and 10 percent for 6 in.

An attempt was made, without success, to develop a similar set of values from Huculak's paper. However, it is possible to obtain a picture of how Canada views the question by reference to a plot shown in a recent publication of the Canadian Good Roads Association, "A Guide to the Structural Design of Flexible and Rigid Pavements in Canada." According to the plot, the reduction of deflection per inch of thickness is dependent on the tolerable or design deflection. For a tolerable deflection of 0. 030 in. the reduction of deflection is about 6 percent per inch assuming the previously mentioned 1:1. 9 layer equivalency for asphalt-concrete to gravel. The reduction increases with increase in tolerable deflection to 12 percent for a deflection of 0. 050 in. As a greater tolerable deflection would demand less thickness, there again seems to be agreement that the highest rate of deflection occurs with relatively thin treatment. The magnitude of the reduction, however, appears less, according to Canadian experience.

At the AASHO Road Test, studies of 99 overlaid (3-in. -AC) sections of the test pavement showed that the deflection, on the average, was reduced 14 percent per inch of overlay, the same value as that cited previously for California.

From the work done in California, as reported by Zube and Forsyth, tolerable deflection limits have been developed. The values vary over wide limits depending on traffic and the makeup of the pavement. According to Figure 2, for pavement having a 6-in. AC surface and carrying heavy traffic the value amounts to about 0. 017 in., for light traffic about 0. 030 in. In contrast, for a 1-in. AC surface, these values are about 0. 040 in. for heavy and 0. 080 in. for light traffic.

At the AASHO Road Test, analysis of deflection data showed that a pavement having a spring deflection of 0. 020 in. would sustain over 6,000,000 applications of an 18,000-lb axle load before its condition dropped to a serviceability level of 2. 5. In contrast, a pavement having a deflection of 0. 060-in. at this time would support only about 200,000 applications before its serviceability dropped to the same level.

Huculak states: "Higher deflections are usually permissible on a lightly traveled highway as compared to a heavily traveled highway for the same magnitude of load." Also, "Some pavements are still in service with deflections as high as 0.075 in. but carrying relatively low volume traffic." The influence of the temperature of AC surfaces is discussed in his report as follows: "Since the effect of lower temperatures is to render the bituminous surface more brittle, the pavement is more susceptible to detrimental cracking during the spring period when the surface layer cannot deform as readily without rupturing." On the overall question of permissible deflections, Huculak is of the opinion that those developed at the WASHO Road Test, 0.035 in. for spring and 0.050 in. for summer conditions, are realistic values for the environment in his areas (Alberta Province).

The problem is complicated because of the interrelated effects of temperature and thickness. It appears that as the temperature of an asphalt concrete decreases and its thickness increases, there would be an ever-decreasing attenuation in tolerable deflection. To a degree this viewpoint is supported by the California data (Zube and Forsyth), i.e., the tolerable deflection level of flexible pavements decreases as the thickness of the AC surfacing increases.

The report of the Canadian Good Roads Association comments on the subject of permissible deflections as follows:

> The performance and life of flexible pavements with rebound deflection values exceeding 0.05 in. is controlled largely by the wheel loads of the traffic. The life of such pavements which carry more than 1,000 vehicles per lane per day, including wheel loads ranging up to 9,000 lb, is drastically reduced as the rebound values exceed this critical value by relatively small amounts.

Also, it is recommended that flexible pavements which will have ADT volumes of 1,000 or more vehicles per lane, including 10 percent or more trucks and buses, within 10 yr after construction be designed for a maximum spring Benkelman beam rebound value of between 0.030 and 0.050 in. The choice of design rebound value within this range depends on the relative costs of initial construction and resurfacing.

Carneiro discusses the question of tolerable deflection in considerable detail, mentioning studies conducted by several investigations in Brazil and abroad, including those of the Road Research Laboratory in England; of D. A. Welsh, British Columbia, Canada; and those of Hveem in California before 1960. He cites the following values being used in Brazil at the present time—0.020 in. for heavy, and 0.028 in. for light traffic—and he implies that they are subject to revision as data are obtained from the extensive work program planned.

The paper of Schrivner, Swift, and Moore describes a unique device for producing and measuring dynamic deflections of a road surface. Since loads on pavements are essentially dynamic in character, it is more realistic to measure deflections under such loads than under standing or slowly moving loads. Certainly the device described is promising; and after more preliminary testing is done and more data are obtained, it should see considerable use in connection with problems of pavement design, performance, and rehabilitation.

This discussion has served to show that there is wide diversity of opinion existing at the present time on the question of tolerable deflection of flexible pavement and, to a lesser degree, on the ability of overlays to reduce deflection. For example, the Zube and Forsyth paper is the only one presenting evidence of the need for reducing the tolerable level of deflection with increase in thickness of overlay. The factor of temperature of the overlay material and its possible marked effect on tolerable deflection is mentioned only by Huculak.

To say that a pavement of the flexible type has a given level of tolerable deflection has little meaning unless its performance for this level is defined. This is emphasized in the findings of the AASHO Road Test, i.e., at a high level of deflection a given pavement section could sustain only a relatively limited number of repetitions of the deflection before its condition deteriorates to the extent of needing rehabilitation; at a lower level of deflection a much greater number of repetitions could be sustained. Data reported by Zube and Forsyth agree with this finding.

Without reference to performance, this writer lists some tentative values of tolerable deflection for different intensities of traffic. Obviously, as data are accumulated it may be possible to relate such values to performances; to put the method of pavement rehabilitation on a firm basis by the use of deflection data, this will be necessary.

There is reasonable agreement among investigators regarding the ability of asphalt concrete overlays to reduce deflection. Data in the California and Brazil papers and those of the AASHO Road Test indicate that 3 in. of the material would reduce the deflection about 14 percent per inch; data in the report of the Canadian Good Roads Association suggest a somewhat lesser contribution for this thickness, about 9 percent. More factual data are also needed on this item; the temperature of the material is obviously one of importance.

In light of this discussion, the following values are offered merely as guidelines for those who may be contemplating the use of deflection data in pavement rehabilitation work. For deflections under 18,000-lb axle load, a tolerable value for light traffic would be 0.060 in., for moderate traffic 0.045 in., and for heavy traffic, 0.030 in. deflection reduction afforded by AC overlay: for a thickness of 1 to 2 in., 20 percent per inch; for a thickness of 2 to 4 in., 15 percent per inch; for a thickness of 4 to 6 in., 10 percent per inch.

For those who expect to continue handling the problem of pavement rehabilitation on the basis of judgment and experience, the following material may prove of interest.

Existing Pavement Condition Category	Method of Treatment
Acceptable riding qualities; minor evidence of structural weakness; some perceptible rutting and some surface cracking.	Thin AC overlay
Acceptable riding qualities; marked evidence of structural weakness; excessive rutting and surface cracking.	3 to 5-in. AC overlay
Inferior riding qualities; minor evidence of structural weakness; some perceptible rutting and some surface cracking.	Leveling course, thin AC overlay
Inferior riding qualities; marked evidence of structural weakness; excessive rutting and surface cracking.	Leveling course, 3 to 5-in. AC overlay
Inferior riding qualities; no evidence of structural weakness.	Leveling course plus thin AC overlay

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•ONLY ONE paper mentioned the measurement of radius of curvature of the deflected surface. This may be measured with reasonable accuracy by means of a very simple and inexpensive device known as a curvature meter which originated in South Africa and is described in Highway Research News No. 19.

Measurements of both rebound deflection and curvature were recently made on 57 projects in Virginia. Correlation between the two indicates that rebound deflection could be estimated from curvature measurements with a standard error of approximately 0.007 in. The uncertainty is greatest on the stiffer pavements of the black-base type where high deflections may be obtained even though curvature values are low. Correlation is much better on the 40 nonblack-base sections where a standard error of only 0.005 in. was computed.

The paper by Scrivner, Swift, and Moore indicates correlation with a standard deviation of approximately 0.007 in. between Dynaflect values and deflections on a variety of pavements. Accuracy might be improved by making separate analyses for specific base types such as untreated crushed stone, asphaltic-concrete, cement-treated aggregate. Although the Dynaflect is certainly a much more costly instrument than the curvature meter, it apparently is more rapid and does not require a heavily loaded truck. Its development should be watched with considerable interest by pavement evaluators.