Pavement Deflections and Fatigue Failures

F. N. HVEEM, Materials and Research Engineer
California Division of Highways

This is a continuation of the paper entitled "The Factors Underlying the Rational Design of Pavements" appearing in the 1948 Proceedings of the Highway Research Board. The original work indicated the importance of fatigue failures caused by resilience in the supporting soils. This paper describes the initial work of measuring deflections over a wide variety of pavements. Examples are shown illustrating the load-deflection curves where pavements are showing signs of failure and on other sections where conditions are good or excellent. In general, the deflections are directly proportional to load, although not in all cases. The deflections were measured under both single-axle and tandem-axle loads and the relationship between these two types of loading are established for several types of pavement.

Laboratory methods are discussed including the design of a resiliometer for measuring the resilient characteristics of soil samples and the design of a fatigue testing machine for measuring the relative flexibility of pavements. The study indicates that a comprehensive design procedure must provide a pavement structure that will either be capable of surviving the fatigue resulting from continuous flexing or have sufficient "stiffness" to reduce the flexing to an acceptable value.

● IN 1948, a paper was presented at the Twenty-Eighth Annual Meeting of the Highway Research Board wherein an attempt was made at identifying and classifying the numerous factors and properties of materials that singly or in combination affect the performance of highway pavements (1). A chart was used to analyze the relationship between the major and minor factors. While the original paper attempted to include all of the factors in the discussion under "Part 1, Analysis of the Pavement Design Problem," the solutions, test methods and design chart proposed at that time were aimed at providing answers to only two of the three primary basic problems shown in Figure 1.

In other words, the design procedures then proposed and which have since been followed in California and subsequently adopted by several other states and foreign countries are confined to anticipating the ultimate density, the amount of moisture which could ultimately be taken up by the soil, and the resistance value of the soil and base material when the worst conditions will have been reached. This procedure, however, does not provide safeguards against failure in the form of cracking or breaking up due to fatigue resulting from continual flexing or bending of the pavement structure under passing wheel loads, Figure 4. This third factor has long been recognized and appears as item three in the second column of Figure 1.

ANALYSIS OF THE PROBLEM

In order to simplify and further illustrate the relationships between the several factors and the structural adequacy of pavements, Figure 2 presents in tabular form the same three basic problems, together with the properties of pavements, bases, and soils which must be recognized and reconciled in order to provide an answer to each problem and thus produce a satisfactory pavement.

In Figure 2 the three problems are listed at the head of Columns 1, 2, and 3, while in the left-hand column the pertinent properties of the basement soils, bases, and pavements are listed in two separate groups. This arrangement is intended to indicate that the engineer must generally accept the basement soil with its inherent properties as it will exist in the roadbed. He must evaluate these properties by suitable tests in order to assign numerical values to the important variables.

It will be noted that the basement soil, whether in situ or imported, is considered to have four important properties which must be determined by separate tests and evaluated independently: (1) internal friction, R-value, measured by the stabilometer; (2)

cohesion, tensile resistance, measured by the cohesiometer; (3) swelling pressure, expansive force exerted during the absorption of water, measured by expansion pressure device; and (4) resilience, compression and rebound under passing loads, measured by resiliometer.

While some selection is often possible the engineer must generally accept the basement soil and deal with the properties as they will exist beneath the pavement after the passage of time, perhaps of several years. However, the engineer has greater freedom in choosing or providing the properties and dimensions of the pavement and the base materials. (The exercise of this choice is often called "engineering judgment.") The most-important of these properties is set forth in Figure 2 as (a) flexural strength, variously referred to as beam strength, slab strength and may be indicated by modulus

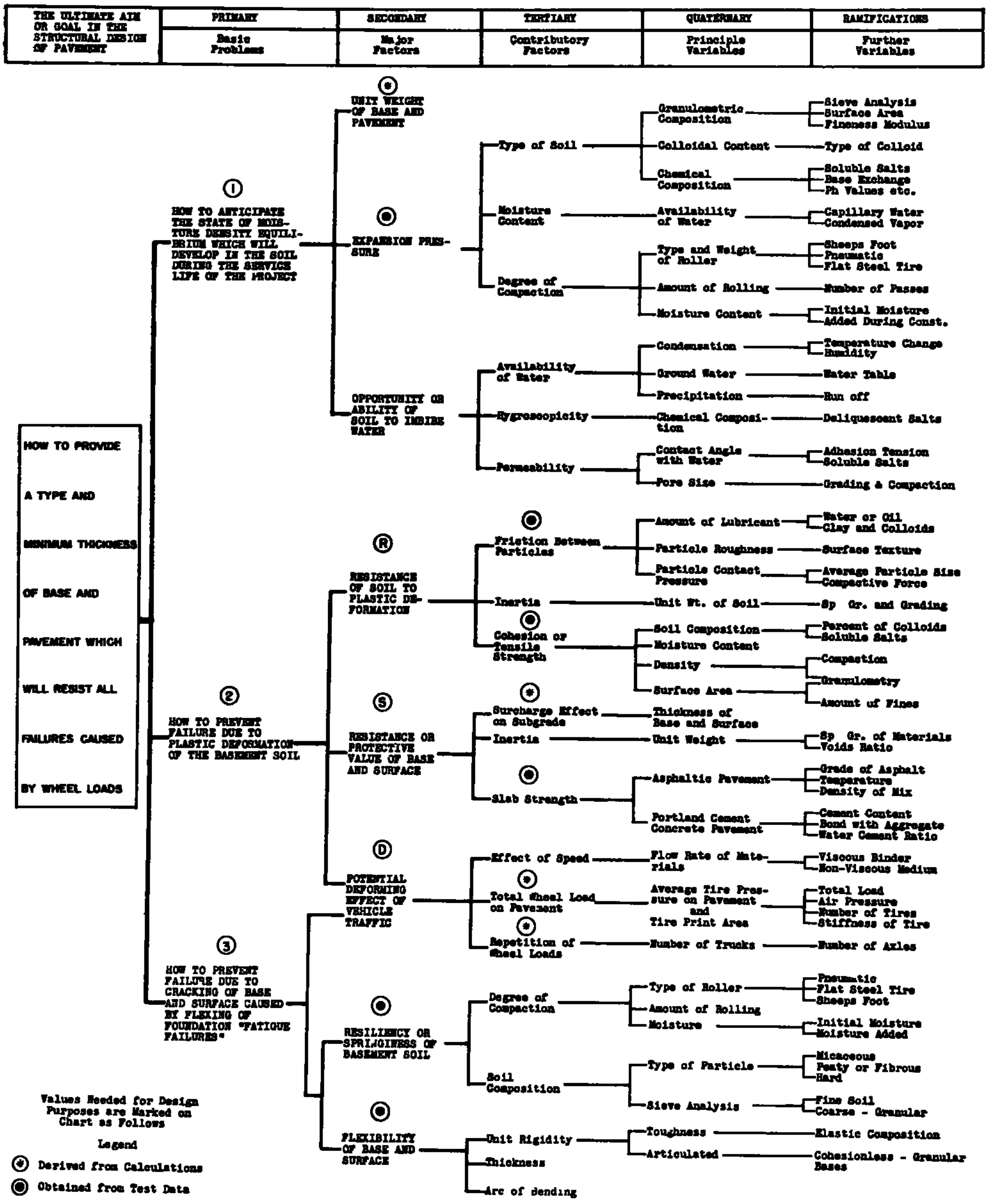

Figure 1. Analysis chart of factors affecting structural adequacy of pavements.

Properties of pavements and subgrades susceptible to measurement which must be taken into account or manipulated in the process of designing adequate pavements and bases		In order to select the type and thickness of pavement and base, three distinct problems must be considered and solved			
		1 EQUILIBRIUM How to maintain equilibrium of moisture and density in basement soil by restraining expansion	2 DISTORTION How to prevent plastic deformation of the basement soil under heavy wheel loads	3 REBOUND How to prevent subgrade rebound from destroying the pavement through fatigue	
Properties	Test Method				
Basement Soil					Factors which must be evaluated to determine the problem
A. Internal Friction	Stabilometer		A. (Major)		
B. Cohesion	Cohesiometer		B. (Minor)	B. (Minor)	
C. Swell Pressure	Swell Dynamometer	C. (Major)	C		
D. Resilience	Resiliometer			D. (Major)	
Pavements & Bases					Factors which may be adjusted or manipulated in solving the problem
a. Flexural Strength	Cohesiometer Beam Tests		a	a	
b. Weight	Thickness Specific Gravity	b	b	b	
c. Flexibility or lack of Brittleness	Test Method being developed			c	

Figure 2. Relationship between fundamental factors governing structural design of pavements and bases.

of rupture values, tensile strength test or by the cohesiometer; this quality contributes to the "stiffness" of the pavement.

The second important contribution by the pavement and associated base layers is (b) direct weight resting on the subgrade, easily determined by computation knowing the proposed thickness and the unit weight of each layer. The "stiffness" factor of the pavement and base combination will also vary with thickness even of granular masses of low cohesive strength.

The third is: (c) flexibility (fatigue resistance), the ability to withstand repeated bending or flexing. The overall flexibility is influenced both by the thickness of the pavement section and by the elasticity. A new device is now under development in the laboratory to evaluate this property in terms of fatigue resistance.

In order to indicate how these various properties may affect the overall performance, in Figure 2 the index letters are transferred and arranged to indicate "answers" in the columns representing the three basic problems. Thus, it will be seen that Problem 1, "How to determine the ultimate moisture and density equilibrium," requires a knowledge of the potential expansion pressure of the soil as it will exist after construction operations; this tendency to expand can only be counteracted by placing a sufficient weight of cover (pavement plus base shown as b in Figure 2) over the soil to balance or oppose the expansion pressure.[1]

The second problem, which is to prevent failure from plastic distortion of the underlying material, requires that the internal friction be measured and, perhaps, the cohesive resistance also; although this latter can usually be neglected for design purposes. The expansion properties are significant here only because they may influence the amount of water taken up and thus indirectly affect the internal friction.

There are two solutions to this problem: (a) to use a pavement of high flexural

[1] Expansive tendencies can, of course, be reduced or eliminated by adding more water during or immediately following construction.

strength; (b) to place a sufficient weight of base and surface, or, as there are no weightless pavements, some combination of the two is employed.

As stated before, procedures for dealing with Problems 1 and 2 have already been set up and are being followed in a number of highway laboratories today. However, so far as is known there have been no organized attempts to deal effectively with Problem 3, which seems to be increasingly serious in recent years due to the great increase in the weight and numbers of heavy wheel loads on highway pavements. Therefore, this paper will deal primarily with pavement deflections under repeated load applications and the resultant fatigue failures.

Referring to the concept illustrated by Figure 2, it will be noted that Problem 3 adds increasing complexities. For Problem 3 it now seems that the internal friction or resistance value of the soil may not be directly significant and cohesive resistance probably plays only a minor part. The main consideration is the actual resilience[2] of the soil in the condition of moisture and density that will be characteristic of the materials after the pavement has been in place for some time.

The lower half of Column 3 indicates there are three possible answers or solutions to the third problem. If a pavement of sufficiently high slab strength is employed (a), then it will not be deflected beyond safe limits by the passing loads. Also, if a sufficient weight or thickness of stable granular material (b) is used in the base course, there will be no undue flexing of the pavement surface. Either or both types may have the required "stiffness."[3] And finally, at direct variance with the limited solutions for Problems 1 and 2, a thin flexible pavement (c) may serve quite well over resilient soils where a heavier, stronger, but more-rigid or brittle type will crack and perhaps show other signs of failure.

However, the materials engineer who must recommend an adequate overall design must make sure that whatever combination of pavement and base is proposed will adequately and simultaneously satisfy all three of the basic problems enumerated. In many cases thin pavements will not satisfy Problems 1 and 2. In view of the fact that methods dealing with Problems 1 and 2 were set forth in the 1948 paper (1) and have subsequently been improved, the following will be confined to a discussion of the factors that must be taken into account for a solution to Problem 3: How to prevent fatigue failures in the pavement due to flexing caused by alternate depression and rebound under moving wheel loads.

DEFLECTIONS

For the purposes of this discussion, the term deflection will be used in a limited and

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[2] "Resilience" is preferred to such terms as elasticity as we are here concerned with movements much greater than would be developed in many elastic solids such as glass, concrete, steel, etc. A new device, termed the "resiliometer," has been developed to measure this property of soils on laboratory specimens.

[3] The term "stiffness" has been borrowed from a report by L. W. Nijboer and C. van der Poel (2). Nijboer computes stiffness from the formula

$$S = \frac{F_p}{X_p} \quad (12) \text{ for } F_p \text{ equal to } 10^4 \text{ N (1 ton) and}$$

2×10^4 N (2 tons) respectively.

F_p = Force acting on pavement

X_p = Deflection of the pavement

Therefore, the term "stiffness" bears a simple mathematical relationship to the deflection of the pavement; as used by Nijboer, stiffness implies the resistance of all components including the pavement, bases, subbases and the underlying soil. For design purposes it seems preferable to us to associate the concept of stiffness with the pavement and base structures alone in which case there will not be a consistent relationship between stiffness and deflection as the character of the supporting soil will then represent a variable: "resilience."

special sense to indicate those movements of the pavement under traffic in the form of downward bending beneath the vehicle wheel followed by rebound after the load has passed on. For this purpose the term "deflection" applies to transient movements and is considered to be only one of several types of deformation which a pavement may undergo.

Figure 3 presents an outline of the terms together with contributory causes which are subdivided into a primary and secondary group. For the purpose of this discussion the following definition applies:

Deflection. A transient downward movement of the pavement when subjected to vehicle wheel loads. A deflected pavement rebounds shortly after the load is removed.

Pavement deflections have been a matter of interest to the writer for many years. While serving as a maintenance superintendent in 1924, prior to the use of bituminous

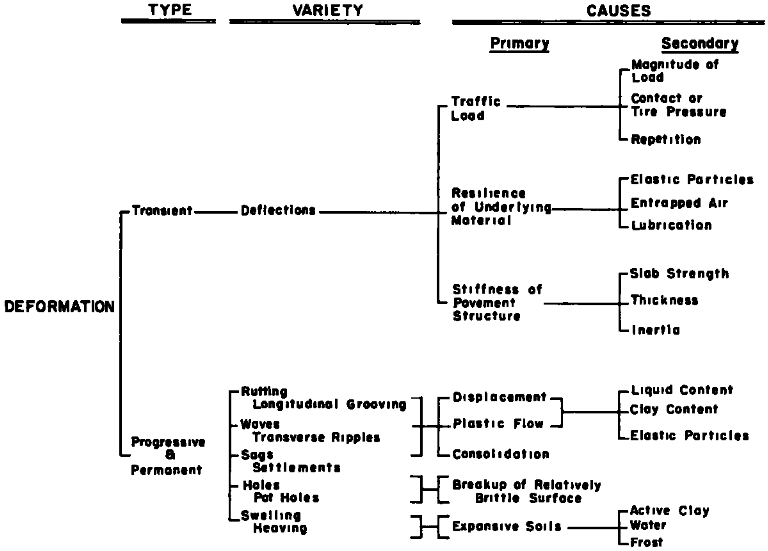


Figure 3. Analysis chart illustrating types of pavement deformation.

surfacings on California's rural highways, he observed that there were marked differences in the difficulty of maintaining untreated gravel roads; in many cases the behavior apparently bore some relationship to the character of the foundation soil. In other words, there were a number of examples on level grades where the gravel surfacing would remain in relatively good condition in cut sections where the roadbed consisted of solid rock as compared to shallow fill sections of fine-grained soils.

The troubles noted were chiefly in the form of raveling and potholing of the surface, but as the surfacing material was the same throughout, it seemed probable that there was a greater magnitude of flexing and bending under heavy wheel loads where the underlying soils were more resilient. Thirty years ago much-less attention was given to the selection of materials in the roadbed, and such material as leaf mold and vegetable matter was not always rigorously excluded as, we hope, is the case today.

In 1938, the laboratory of the California Division of Highways secured a General Electric travel gauge to measure deflections of pavements under rapidly moving wheel loads. This unit was used in scattered investigations throughout the years both on state highways and for test pavements constructed by the state and the Corps of Engineers. It was found nearly 15 years ago that asphaltic pavement deflections varied greatly with temperature. However, it was not until 1951 that an organized study was undertaken to determine the actual deflections that traffic was inflicting on California highway pavements. The data furnished herewith are intended to be a progress report of a study which is by no means completed. The data represent selected examples from 43 projects involving the installation of nearly 400 gauge units and over 2,500 individual gauge records.

In 1951, a newly constructed section of asphaltic-concrete pavement (less than 2 years old) was showing marked evidence of distress in the form of extensive cracking of the type usually described as "chicken-wire" or "alligator" cracking (Figure 4). This section of road is a four-lane divided highway north of Los Angeles, on US 99 which is the principal truck route between Los Angeles and points north. The cracking was first observed and became most pronounced in the outer lane, which carries over 80 percent of all traffic and virtually all of the heavily loaded trucks. However, on a short stretch of this project all lanes of the pavement remained in good condition with no evidence of cracking.

Deflection gauges were installed in both cracked and uncracked areas, and the deflection of the surface was measured with reference to rods driven through the base and anchored in the underlying soil at various depths. Figure 5 illustrates a typical installation of these gauge units. Figure 6 is a plot of the deflections which were measured on this project against reference rods 3 feet long, using axle loads ranging from 11,000 to 29,000 lb. as indicated by the abscissa scale.

It will be observed that there is a marked difference in the magnitude of the deflections measured where the pavement is badly cracked compared to the area where there is no evidence of cracking.

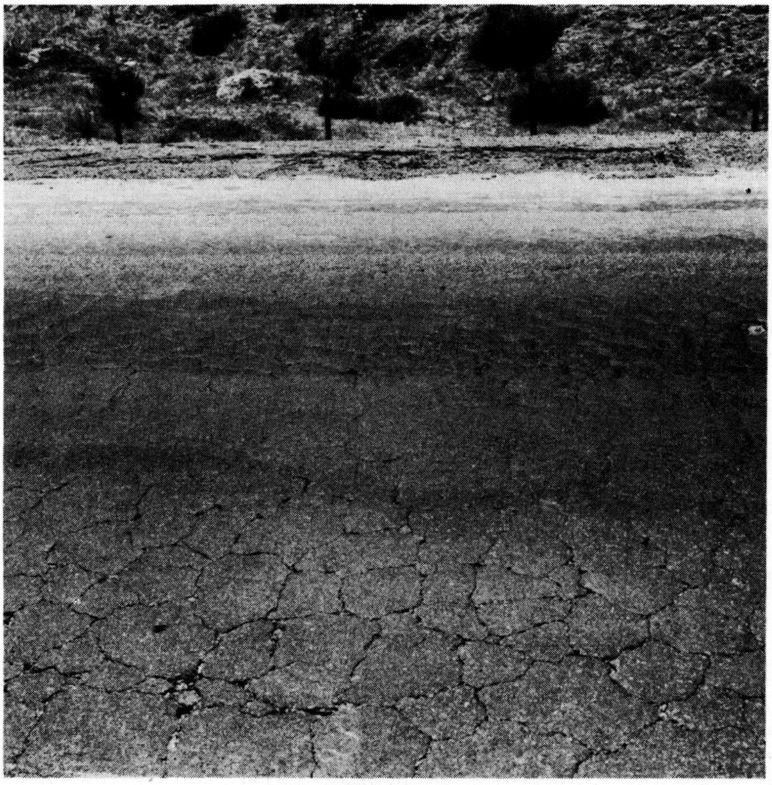


Figure 4. Typical illustration of "chicken wire" or "alligator" cracking.

Most of the deflection measurements that have been made to date indicate that in general the measured deflection bears a linear relationship to the load applied, although this relationship does not everywhere hold true. As will be shown later, certain types of soil (especially where deflections are high) develop a distinctive load-deflection pattern that is not in accord with the linear relationship indicated in Figure 6. In any event, it will be apparent that the badly cracked portions of the pavement have been continuously subjected to much higher deflections than has the section where no cracking is in evidence. It might be argued, of course, that the deflections are higher because the pavement has cracked and thus lost continuity and slab strength. Undoubtedly, the deflections are greater when the slab continuity has been destroyed; however, the de-

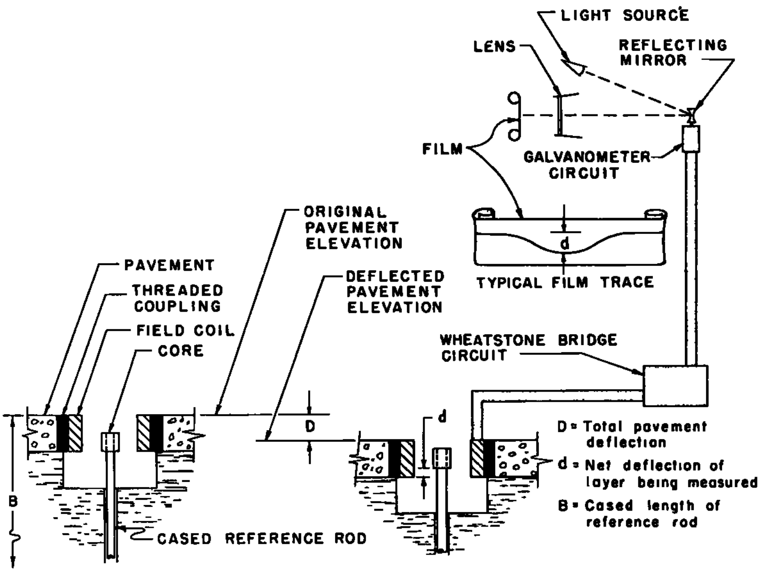


Figure 5. Schematic diagram of apparatus for deflection measurements.

flection in the passing or inner lane at the same location is also relatively high compared to most uncracked pavements, and the absence of cracking in the inner lane can only be attributed to the fact that it carries relatively few heavy loads.

The structural section used on this project is shown in Figure 7. It will be noted that it is as heavy and substantial as that used on the New Jersey Turnpike, for comparison. The asphaltic-concrete surface has a stabilometer value of 45. The crushed granite base has an R-value of 77 and a CBR of 161, the subbase has an R-value of 74 and a CBR of 125. However, it now appears that the basement soils on this project are definitely resilient, and there is also evidence that the asphalt has become hardened; therefore, the pavement is more brittle than desirable.

The original asphalt had a penetration between 120 and 150; however, recoveries of the asphalt made in 1951 by the Abson method show an average penetration of 36 after one year on the road. The records indicate that plant temperatures were quite moderate and unusually well controlled ranging between 275 and 285 F. While subjected to heavy

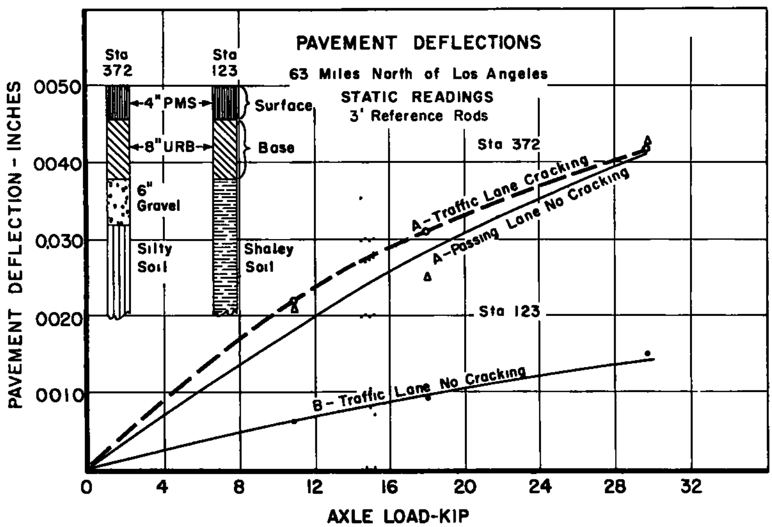


Figure 6.

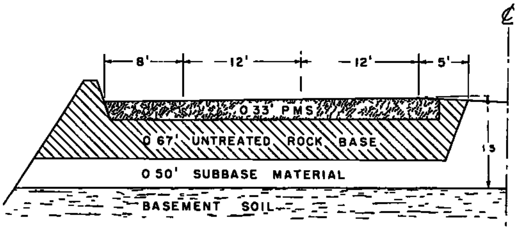


Figure 7. Typical cross section road VII-LA-4-J, Station 372+20 pavement cracked.

traffic and badly cracked, as noted above and shown by Figure 4, this pavement has remained smooth and undistorted, indicating that the so-called failures were entirely due to flexing and bending.

The second project that was investigated is on the Bayshore highway, US 101, one of the heaviest-traveled routes in the state, being the main traffic artery from San Francisco to the south. A portion of this highway was reconstructed on new alignment in 1947, traversing an area of mud flats which required imported embankment materials up to 25 feet in depth surfaced with 5 inches of asphaltic concrete resting on 8 inches of crushed stone over 24 inches of sand subbase.

Failures in the asphaltic surfacing in many areas on this section (Figure 8) became evident within 1 or 2 years after construction. As an investigation conducted by the laboratory could discover no deficiencies in the quality of the asphaltic pavement or of the base material, it was decided to measure the magnitude of deflections. Figures 9, 10, 11 and 12 illustrate the deflection measurements made on this project using reference rods of different lengths.

Figures 9 and 10 represent the deflections at two locations where the pavement is in good condition.[4] Figures 11 and 12 represent readings taken at points where cracking and distress of the asphalt pavement were evident. Figure 13 gives a profile view of the measured deflections illustrating the length of pavement involved in these deflection zones.

Attention is drawn to the evidence that truck loads can affect the pavement foundation to a depth of 18 feet or more. It will be obvious, of course, that the magnitude of vertical deflection is not of itself completely significant, as the tendency to break or rupture the pavement will depend primarily upon the sharpness of the arc or curvature of the pavement surface.

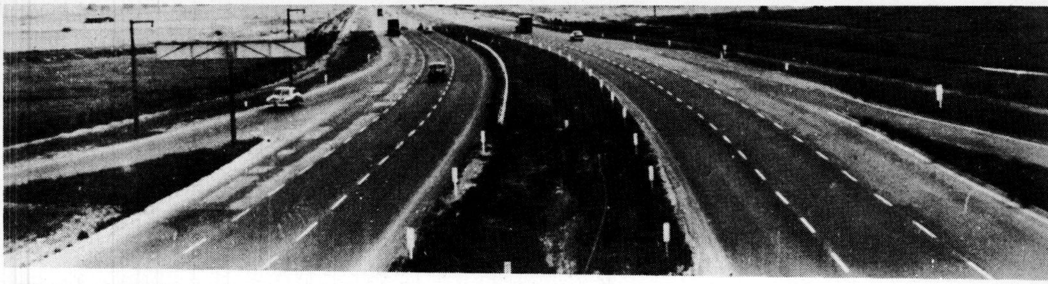
Engineers in Sweden have concluded that when the pavement is bent in an "arc" having a radius less than 100 feet, failures would result. It may be that asphalt pavements in Sweden are more flexible; in any event we have been unable to arrive at a similar value for California conditions, although the general idea seems sound.

While the vertical deflection measured with reference to a rod 18 feet in depth is of the greater magnitude, it may well be that the most-severe stresses in the pavement are associated with compression and rebound in the upper layers of the embankment, which may produce sharper bending and consequent greater stress in the pavement slab. In other words, the shape of the depressed area will undoubtedly vary with variations in both basement soil and pavement structure.

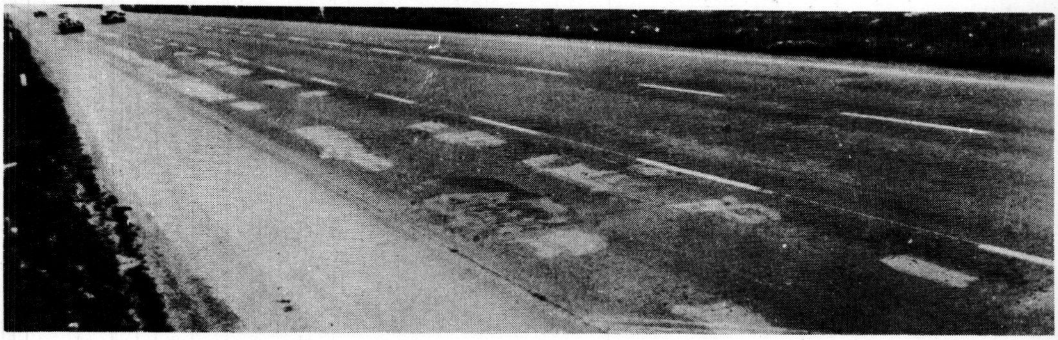
It must also be recognized that a moving wheel load causes a sharp reversal of stress from tension to compression in every portion of the pavement in the wheel path. There is evidence to indicate that in many cases the sharpest bend and consequently the greatest stress is outside the wheel contact area.

After the initial studies outlined above, it was tentatively assumed that the compressibility and rebound in the top 8 or 9 feet of the roadbed soil would be of greater significance and more readily correlated with performance than the total overall deflection.

[4]One of these was in an area where vertical sand drains had been placed in the embankment. This poses an interesting question about the function and performance of this currently controversial type of installation. There is no evidence that the fills have settled more rapidly with the sand drains than adjacent areas without this provision. Borings alongside the sand columns brought no evidence of dewatering of the soil. Nevertheless the pavement is in better condition and as stated above the deflections are markedly lower. Perhaps the 3-foot layer of pervious sand placed as a blanket over the vertical drains is the answer.



Sta. 395± General Outside Lane Failure



Sta. 498± General Outside Lane Failure

Figure 8. Typical failed areas. San Francisco Bayshore Freeway.

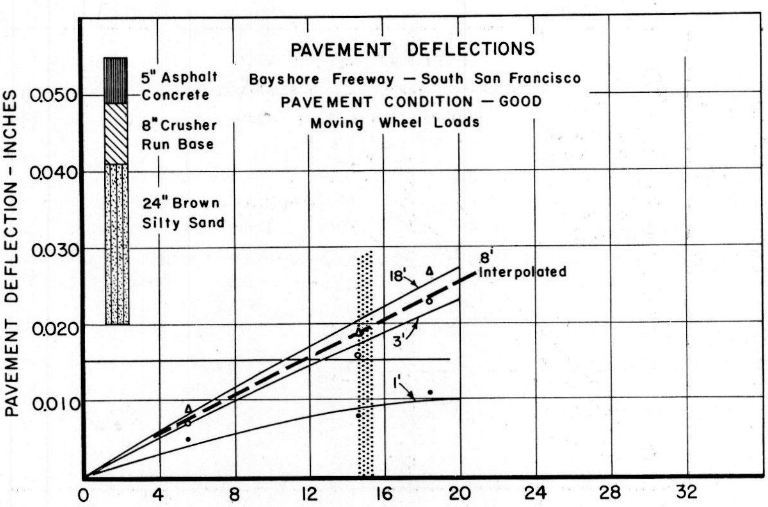


Figure 9.

Therefore, on the more-recent work, these depths have been used for purposes of comparison, although it is readily admitted that the question of how best to employ a simple measurement of deflection as an index of destructive bending movements has yet to be settled in our minds. However, in order to "start somewhere" we have compared deflections referred to an 8-or 9-foot reference rod under 15,000-lb. single-axle loads. This means that what we are actually reporting is the compression and rebound of the soil in the upper 8 or 9 feet of the roadbed.

An asphaltic-concrete pavement on US 40 between Sacramento and San Francisco has given an excellent performance since 1937. Deflection measurements were made as shown in Figure 14 on a section that was definitely in good condition. Here it will be noted that the compression and rebound in the top 9 feet of the roadbed under a slow-moving 15,000-lb. axle load is 0.012 inch. In this instance we have also shown, for comparison, the deflection caused by a static or standing load. While the difference here is greater than most, it is true that deflections under static loads are always greater than under moving loads.

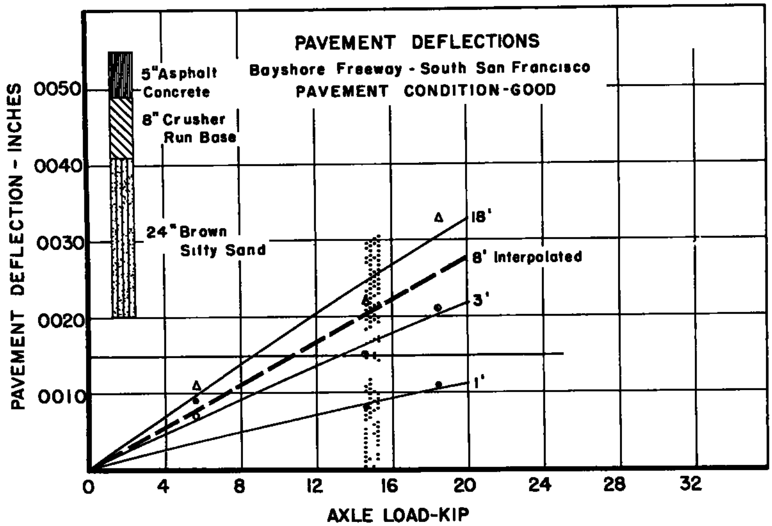


Figure 10.

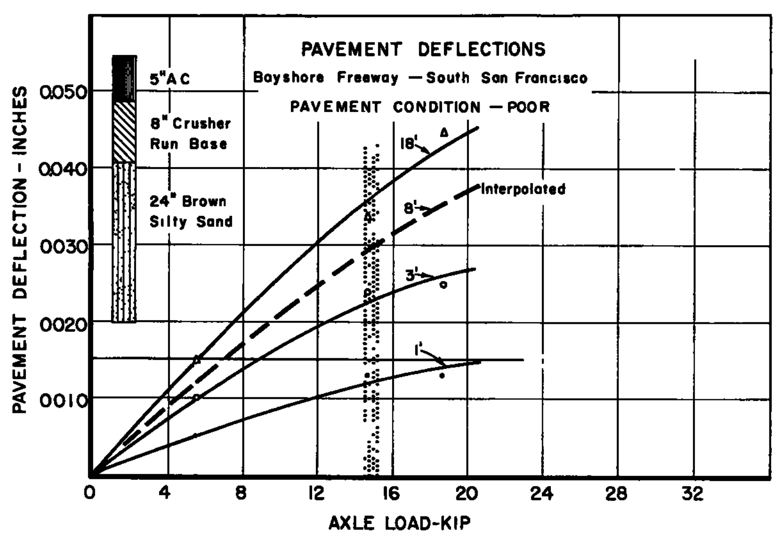


Figure 11.

By way of comparison, Figure 15 refers to a portland-cement-concrete pavement 5 miles north of Eureka. This pavement is some 28 years old and in fair condition, considering its age, type of foundation, and amount of traffic. Here the deflection related to the upper 9 feet is moderate, being 0.016 inch.

Next, a few asphaltic surfaces were studied, on the Redwood Highway where the road is subjected to heavy log hauling. One section across swampy ground (Beatrice Flats) has undergone periodic settlements throughout the years. As a result, the roadbed has been built up by additional layers of gravel and bituminous surfacing, until at the present time the total thickness of gravel is over 30 inches. The asphalt surface is now in excellent condition and has remained so for some time. Again one may note from Figure 16 that the deflection with reference to a 9-foot rod is 0.017 inch.

An even-more-striking example is the present road on the Redwood Highway (south of the town of Scotia) which was reconstructed in 1946 using a 3-inch plant-mix surface over an 8-inch cement-treated base supported by a substantial subbase of pit-run gravel. Here the deflections under a 15,000-lb. load for a 9-foot depth are only 0.009 inch (Figure 17). The excellent appearance of this surface and freedom from maintenance cost over a period of 8 years testify to the fact that this pavement is adequate for the heavy traffic which it must sustain.

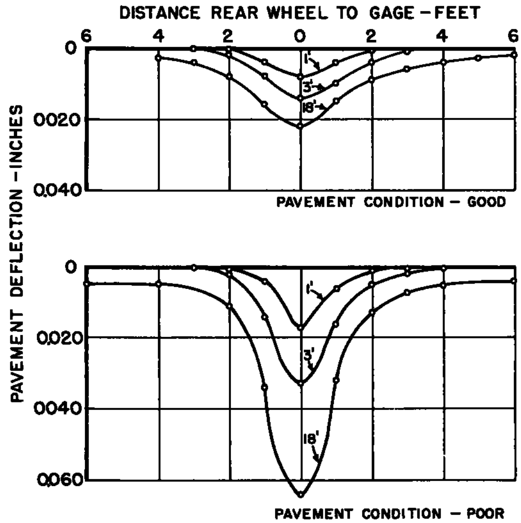


Figure 13. Pavement deflections, Bayshore Freeway, South San Francisco, 14,000 lb.-wheel load.

In marked contrast are the high deflections measured on a section of old secondary road 21 miles southeast of Eureka, where the old Warrenite pavement shows evidence of extensive cracking. This section has been resurfaced several times, and the maintenance costs have been high. Figure 18 illustrates deflections as high as 0.140 inch.

One might comment at this point that damage to a pavement is not necessarily in direct proportion to the magnitude of the deflection. Once the pavement is cracked into small blocks, it then acquires the ability to bend, and this "articulated" structure can accommodate considerable movement without necessarily progressing rapidly to complete failure. Such cracked pavements are a worry to the engineer; however, in many cases they will carry traffic for a long time, proving that true flexibility is a desirable characteristic.

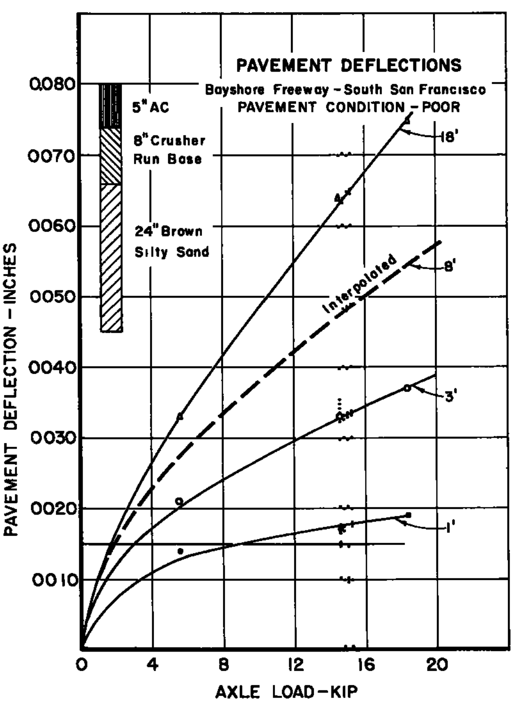


Figure 12.

Figure 19 shows a comparison between the deflections measured in a pavement supported in one area by a cement-treated base and in another by a gravel base. The gages were set in the outer wheel track of a traveled way, supported by 6 inches of cement-treated base. Gages were also placed in the adjacent shoulder section, surfaced with plant-mix resting upon 19 inches of pit-run sand and gravel containing appreciable amounts of clay and silt. Here it will be noted that the cement-treated base apparently has a marked effect in reducing the pavement deflections.

Additional confirmation of the effect of slab strength is shown by Figure 20, which illustrates the low deflections of a section where 4½ inches of plant-mixed surfacing is supported by 8 inches of cement-treated base. This section is about 43 miles north of Los Angeles on the main highway known as the Castaic Bypass. The appearance of this pavement is excellent, with only an occasional transverse shrinkage crack. The use of cement-treated bases appears to be an effective means of reducing the deflections in many cases.

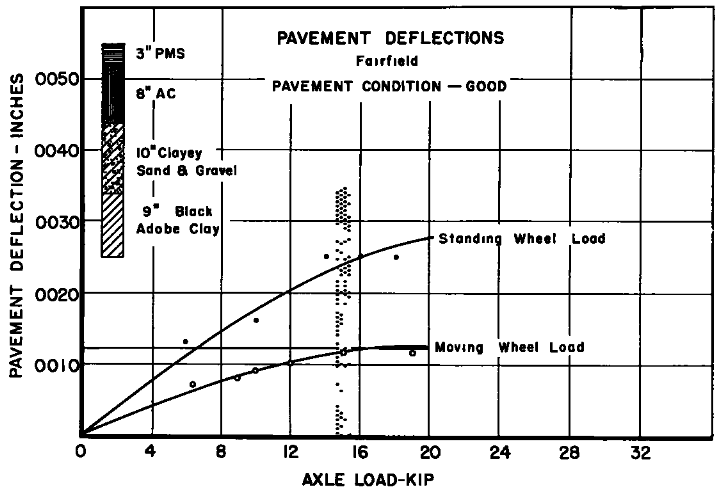


Figure 14.

However, there is also evidence that deflections may be reduced to equally acceptable limits by means of heavy gravel or crushed-stone bases, as is illustrated by Figure 16 and Figure 21. Figure 21 shows the same comparison as Figure 19, but in this case the difference in deflection between the pavement resting upon a cement-treated base and the adjacent shoulder supported only by sand and gravel is small. The excellent condition of both shoulder and traveled way are in accord with the low deflections measured.

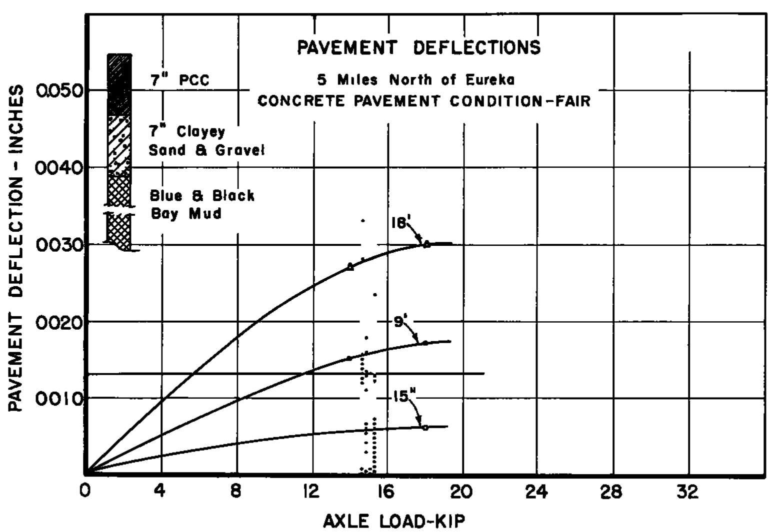


Figure 15.

A number of deflection measurements have also been made on concrete pavements, most of which are in relatively good condition, however. Some of these curves showing deflection versus load are arranged in Figure 22. It is a difficult matter to make comparisons between the deflections of a concrete pavement and those of a bituminous type.

The deflections will vary throughout the length of the average concrete slab, which at night or in the early morning is usually curled up at each end, thus losing contact with the subgrade. The deflections, therefore, are generally greater at the ends of the slabs than at the midpoint, and this curling or warping is affected by both temperature and moisture. Therefore, in order to measure deflections which reflect the bending of the slab due to compression and rebound of the subgrade, it is necessary to place the gages in the slab midway between the joints. In any event, it will be noted that the deflections are all comparatively low for concrete pavements in good-to-excellent condition.

It must be pointed out, however, that the ends of most concrete slabs are being continually flexed under passing vehicles, not because of subgrade compression but because the ends are often unsupported for a distance of 5 to 7 feet from the joint. Therefore, failure and breakup of concrete pavements may, in many cases, be unrelated to subgrade compression and rebound. Obviously, the problem of measuring and evaluating pavement deflections refuses to remain simple.

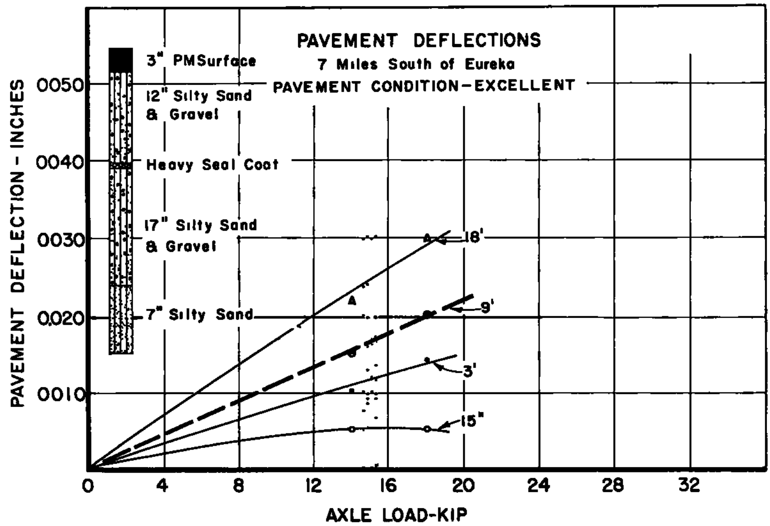


Figure 16.

As an aid in visualizing the shape of the depression "basin" in the pavement, a three-dimensional model was carefully constructed to an exaggerated scale. Figure 23 is a photograph of this model, representing a typical deflection pattern of a bituminous pavement on a gravel base under a 9,000-lb. dual-tire wheel load.

PAVEMENT CONDITION VERSUS DEFLECTIONS

A few examples were found where pavements undergoing fairly high deflections appeared to be in good condition, as indicated by some of the solid lines in Figure 25, but it will, of course, be obvious that asphaltic pavements inevitably vary somewhat in their ability to withstand repeated flexing without evidence of cracking, (Figure 32). There are differences in the aggregate gradations, in the grade and amount of asphalt. There are differences in ages of the pavements, in the thickness and differences in the climatic temperature range.

As mentioned above, most of the deflections shown thus far have indicated an approximately linear relationship with load; that is, the magnitude of the deflection is in direct proportion to the magnitude of the load. However, in one area of the state (on the coastal highway near San Luis Obispo) marked cracking of the pavement has been observed on two separate contracts separated by a few miles but utilizing similar materials in the imported subbase layers.

A plot of all deflections measured on these sections shows a characteristic curved pattern, (concave downward) indicating that the deflections are disproportionately high for the lighter loads. Nevertheless, it is true here, as elsewhere, that the measured

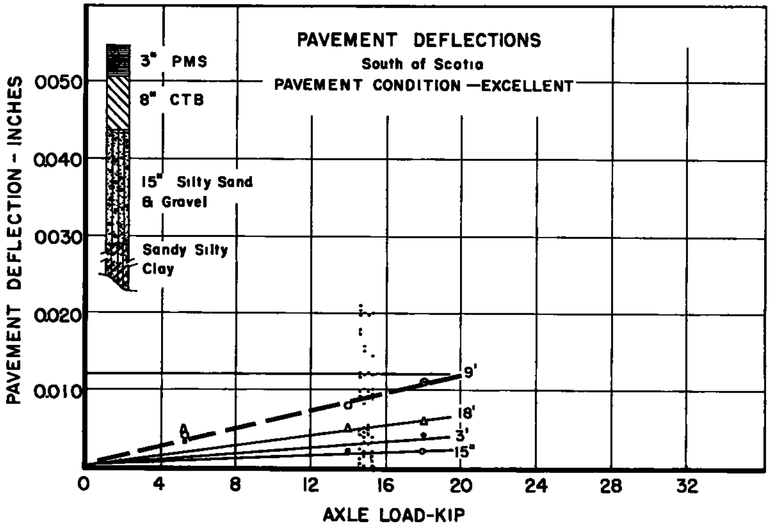


Figure 17.

deflections under 15,000-lb. axle loads in areas where the pavement is in good condition are generally less than 0.020 inch. Deflections made in cracked and failed areas on these sections generally exceed 0.025 inch. Figure 24 shows deflection measurements on this section.

The data shown herewith represent only selected examples of a large number of readings that have been made over California pavements. Figure 25 is a summary chart showing a comparison between the deflections characteristic of cracked pavements compared to those found where the surfacing is in good or excellent condition.

Deflection measurements have been made by the Corps of Engineers on airport pavements and by the Bureau of Public Roads and the Highway Research Board on the experimental test tracks of Road Test One-MD in Maryland(3) and on the WASHO track in southern Idaho (4). There has not been an opportunity or time to compare all of the available deflection data in order to establish general laws or rules. Knowing something of the variations which may exist in asphaltic paving mixtures, it would be unwise to make too-positive statements at this time concerning the amount of deflection which an individual pavement in a given area and climatic environment may safely withstand. However, considering that the study is incomplete and that further evidence may cast a different light on these conclusions, it now seems clear that the type of dense-graded asphaltic pavements frequently constructed in California (approximately 3 inches thick) will not long endure repeated flexing that exceeds 0.020 inch; heavier pavements appear to be limited to even lower values.

It must be pointed out that this study does not undertake to say exactly how much deflection is being produced by the

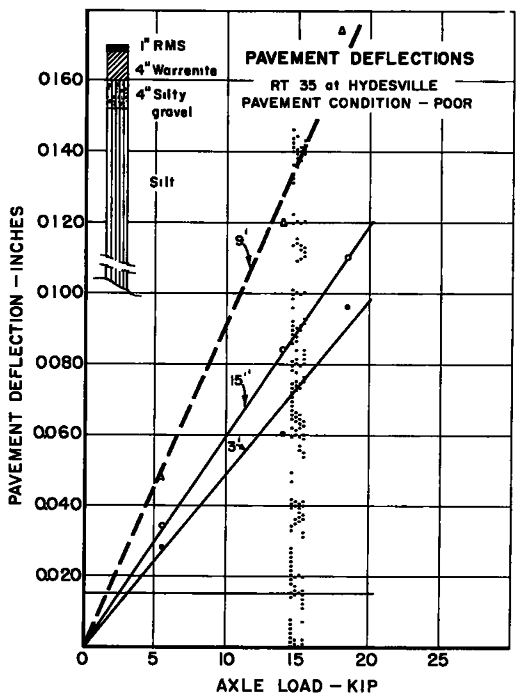


Figure 18.

current truck traffic passing over the road. As the failures are a fatigue phenomena, cracking is the result of both the magnitude of bending, or flexing, and the number of repetitions.

Most highway traffic represents a distribution or range of loads; while a moderate number may reach or even exceed the 18,000-lb.-axle-load limit, it is evident that the "average" wheel load must be somewhat less. Therefore, we have a complex problem in evaluating traffic where a few heavy loads cause high deflections and the many lighter loads cause lower but more-numerous bending repetitions.

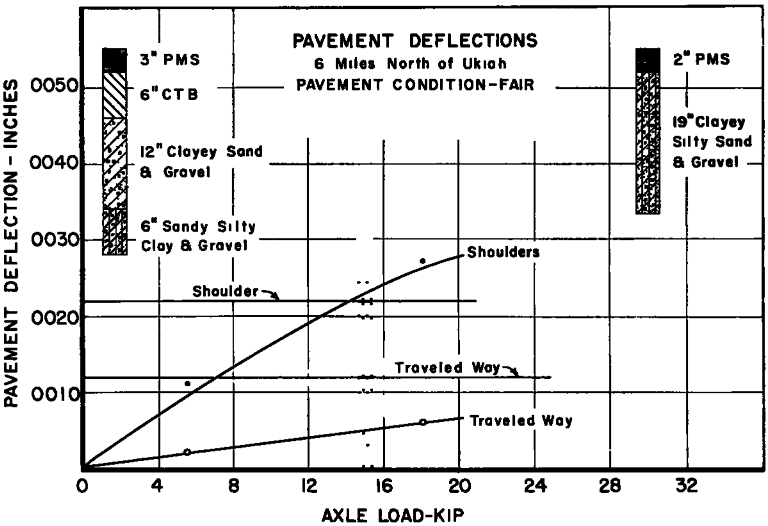


Figure 19.

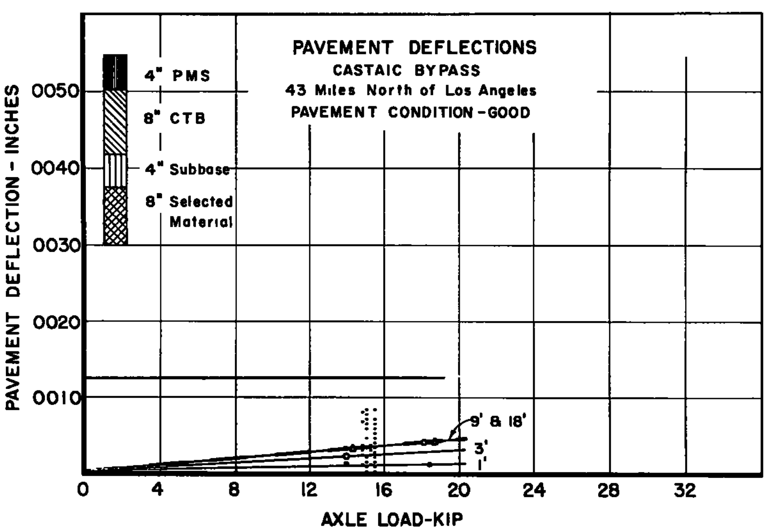


Figure 20.

We have assumed that an overall summation would be equivalent to an equal number of repetitions of axle loads of about 15,000 lb. On this assumption the cracking and fatigue failures of most pavements are attributable to a large number of deflections having an effective equivalent greater than 0.020 or 0.025 inch. Thinner or more-flexible pavements would obviously raise this limit of tolerance; or in the absence of heavy loads, the pavement would not be subjected to the magnitude or number of bending stresses.

VIBRATIONAL METHODS VERSUS DEFLECTIONS UNDER SLOW MOVING LOADS

Thus far we have discussed pavement deflections produced under slow-moving truck loads and the apparent relationship between these deflections and the condition of the pavement surface. The study has many interesting ramifications that have not been touched upon. For example, we were able to undertake some comparisons between the deflections caused by slow-moving loads on pneumatic tires and those developed by vibrational means.

Through the courtesy of the Shell Oil Company, the vibration tester developed in Holland (2) was made available, and measurements of strain, deflections, and velocity of wave propagation were taken during July 1954 at a number of locations on California highways where electronic gages had been previously installed and earlier readings secured. Attempts to establish a correlation between the deflections under wheel loads and those produced by the vibration machine were not too successful, probably for several reasons: (1) the lapse of time between the GE gage readings and the measurements with the vibration machine and (2) the fact that the vibrator operates through a heavy superimposed dead load, which probably tends to suppress some of the amplitude of movement.

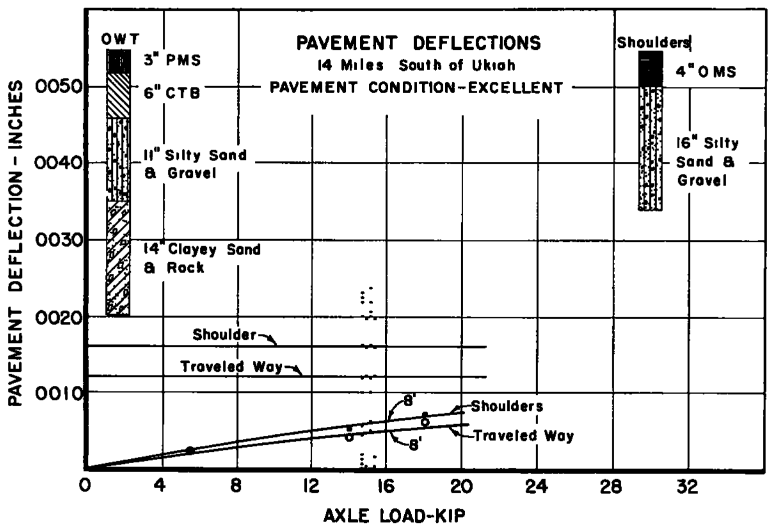


Figure 21.

Some comparison was made between deflections indicated by the Shell vibrator and those obtained by the Benkelman beam (4) on the same day. While it is apparent that no close correlation exists, there is a general relationship, even though the range of values obtained by the Benkelman beam under a 9,000-lb. wheel load is greater than the dynamic deflections registered by the vibration machine developing a force equal to 2 metric tons. Figure 26 shows a Benkelman beam being used to measure deflections caused by a heavy wheel load.

In general, it appears that there is no major difference in the evaluation of pavement stiffness which would be arrived at by either of the two methods, and the low cost, simplicity and speed of operation with the Benkelman beam device makes it an attractive instrument for the initial study of pavement deflections. The Benkelman beam does, have the limitation that it is impossible to identify the layer of material beneath the pavement that is responsible for such deflection as may occur. For this purpose we have found no substitute for the electric units, which make it possible to install a series of reference rods of varying lengths and thus identify the layer or horizon beneath the pavement that is chiefly responsible for the compression and rebound action. The GE gages also avoid a possible error that might result with the Benkelman beam where the length of pavement depressed is greater than 8 feet (Figure 13, for example).

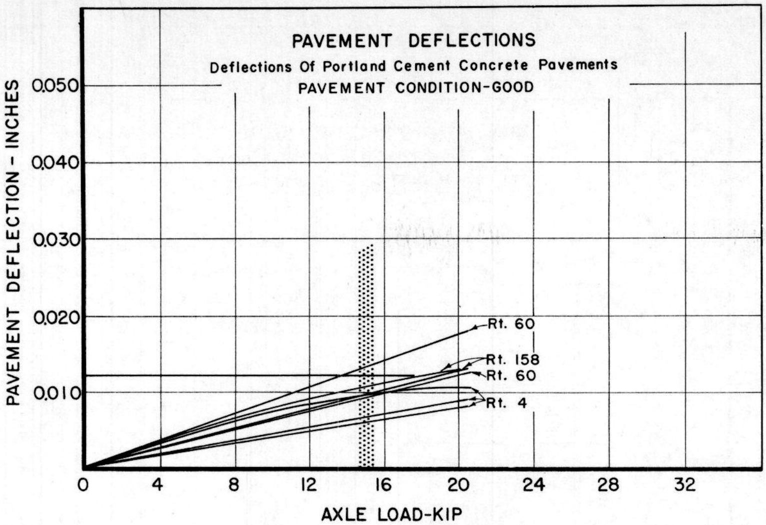


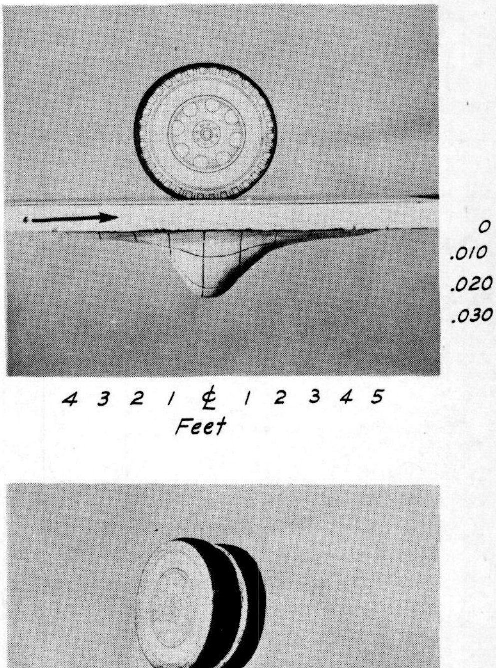
Figure 22.

Work in this laboratory has not progressed far enough to permit setting up a laboratory procedure giving information which will enable the designer tó anticipate conditions of resiliency in the basement soil. Work on these lines is under way and we now believe we are justified in feeling optimistic about the outcome.

NEW TESTS AND DESIGN PROCEDURES

It appears that there are three major subdivisions of the laboratory work and the analytical steps necessary to develop a solution for this problem: First, is the measurement of deflections which are characteristic of existing pavements. This investigation requires many measurements over as wide a variety of pavement types and conditions as possible. From this study it should be possible to establish the magnitude of deflection which is characteristic of the failed sections compared to the amount shown by pavements in good condition. This work is under way in California and some of the preliminary results have been discussed and illustrated in the first part of this report.

In order to utilize the findings in the daily work of highway and airport engineers and to develop more realistic designs, it will be necessary to have laboratory means for evaluating the potential resilience of soils and proposed foundation materials and to be of any use for the average highway program such tests must be performed



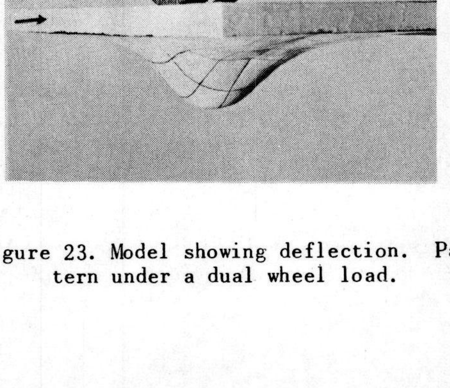


Figure 23. Model showing deflection. Pattern under a dual wheel load.

on samples taken in advance of design and, of course, even further in advance of actual construction.

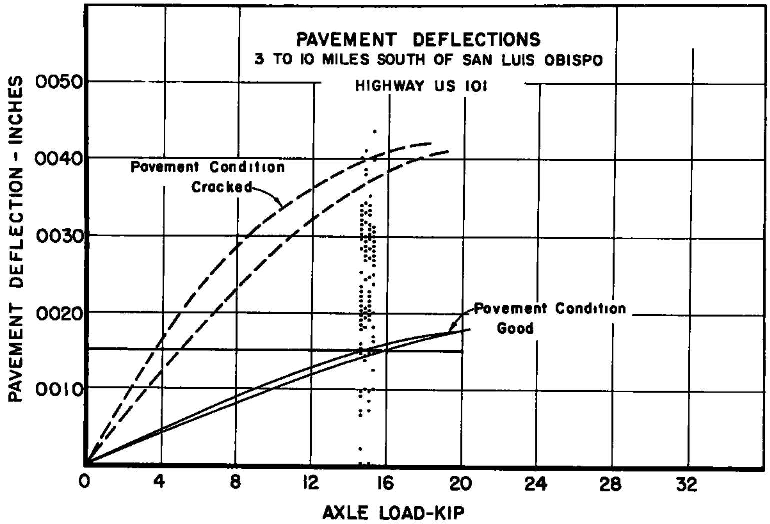


Figure 24.

MEASURING RESILIENCE OF SOILS

The first model of a resiliometer was developed and constructed in this laboratory in 1946. Preliminary trials indicated that it was possible to measure differences in the compression and rebound characteristics of soils, but work on the device was side-tracked for a time due to pressure of other projects. An improved model was constructed in 1954, and work was well under way until interrupted by a fire in the laboratory in March 1954. Since that time, resiliometer Model 3 has been designed and constructed (Figures 27 and 28). For the first time results appear to be consistent, and it now seems that it will be possible to measure and evaluate potential resilience using the small standard stabilometer specimens 4 inches in diameter and 2½ inches in height.

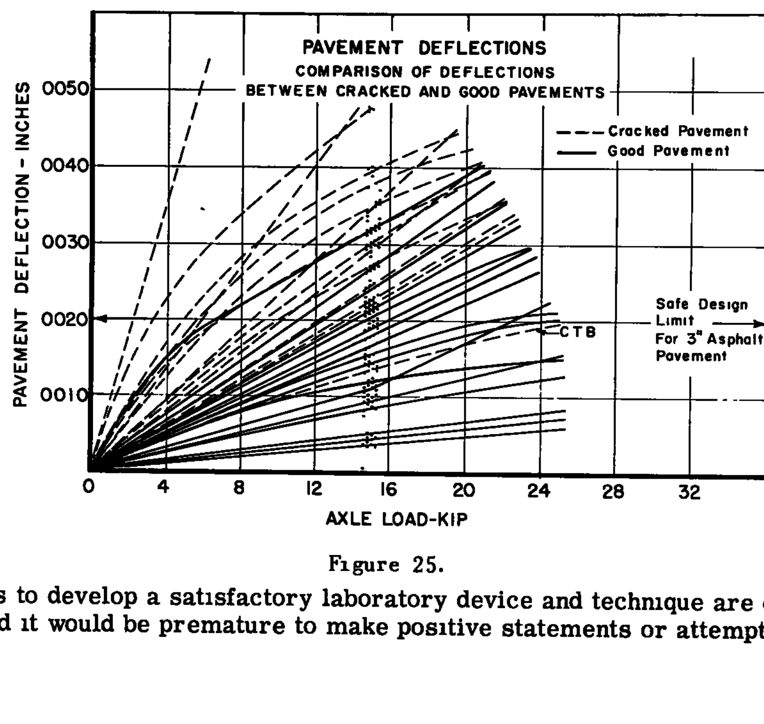


Figure 25.

Attempts to develop a satisfactory laboratory device and technique are only well started, and it would be premature to make positive statements or attempt final conclusions at this time. For example, it is not yet clear what pressures should be used in the resiliometer cycle in order to subject specimens to forces of the same order of magnitude as the pressures transmitted to the basement soils through the overlying layers of base and pavement.

Obviously, of course, in the actual roadbed the pressures will vary with depth; but for practical routine testing purposes, it would be much-more convenient to deal with a single figure for resilience using a single standard pressure in the test apparatus.

Tentatively, therefore, we think that a pressure of 20 psi. may be about right. Figures 29 and 30 show readings obtained with the resiliometer. Resiliometer readings are in terms of the volume of displacement or compression and have not yet been correlated with linear units of pavement depression.

Figure 29 is an expansive resilient soil from southern Idaho. This graph illustrates vividly that resilient properties are not manifest until the voids in the soil are filled with water, after which the susceptibility to compression and rebound increases rapidly with further increase in the moisture content. It is easily demonstrated, of course, that an expansive soil will take up moisture well beyond the point usually referred to as "maximum density"and "optimum moisture content."

Figure 30 illustrates resiliometer measurements on samples of well-graded gravel. Here the addition of moisture tends to diminish even the small amount of resiliency that exists.

While several details of technique and laboratory procedure are still to be settled, these results seem to warrant the belief that a successful test procedure can be evolved. It will be observed that the magnitude of deflection and rebound increases with increasing moisture content, after a certain value has been exceeded, and also increases with increasing unit pressure. These preliminary results strongly suggest that the flexing of pavements under passing wheel loads may undergo a sharp increase in magnitude as soon as the subgrade moisture reaches the saturation point. This is especially true of the agricultural soil types containing appreciable amounts of fine materials or clay and probably entrapped air.

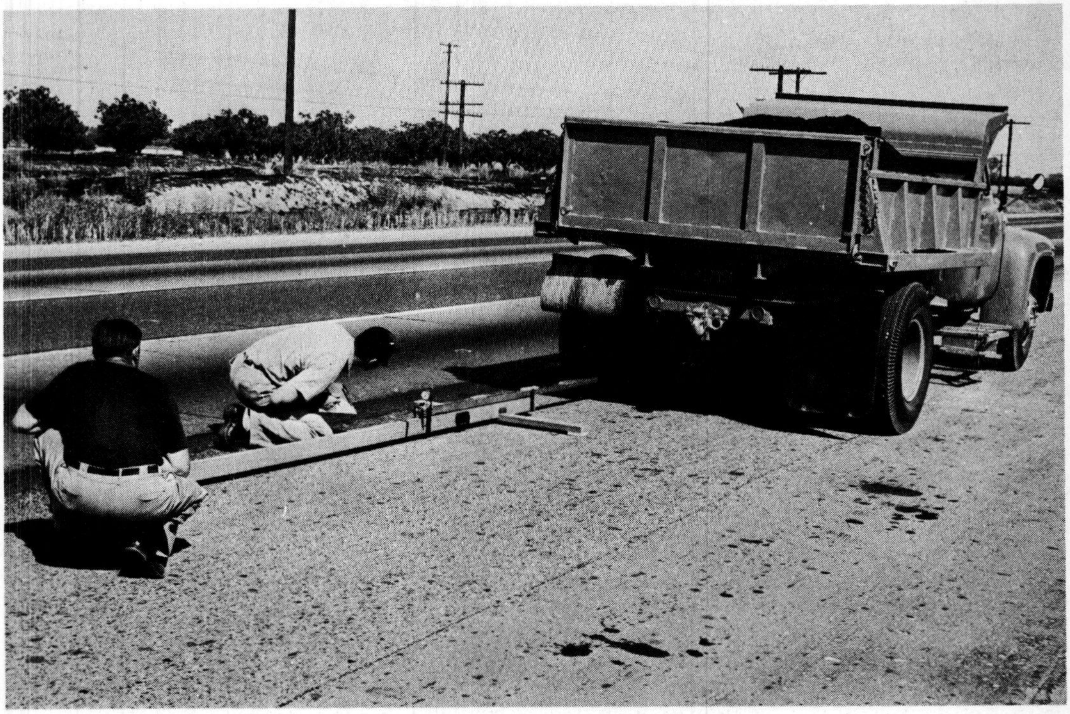


Figure 26. Benkelman beam being used.

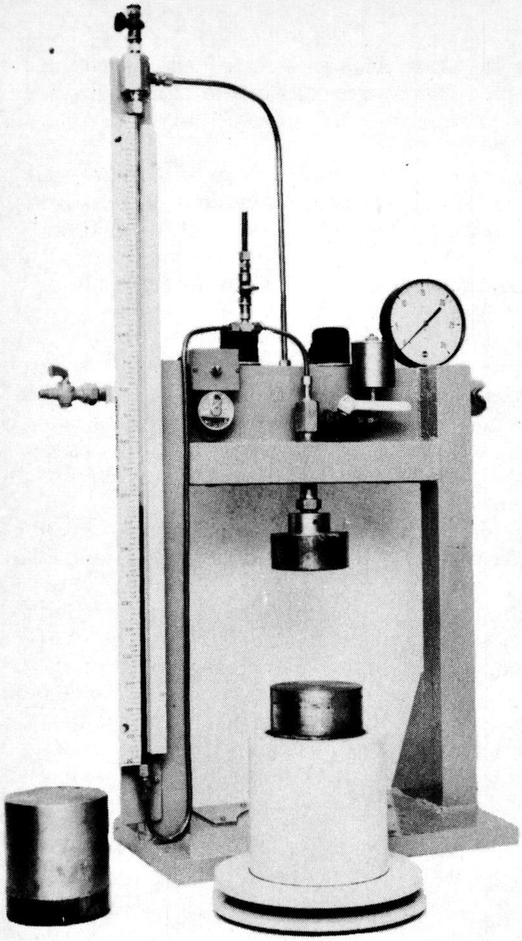


Figure 27. Resiliometer.

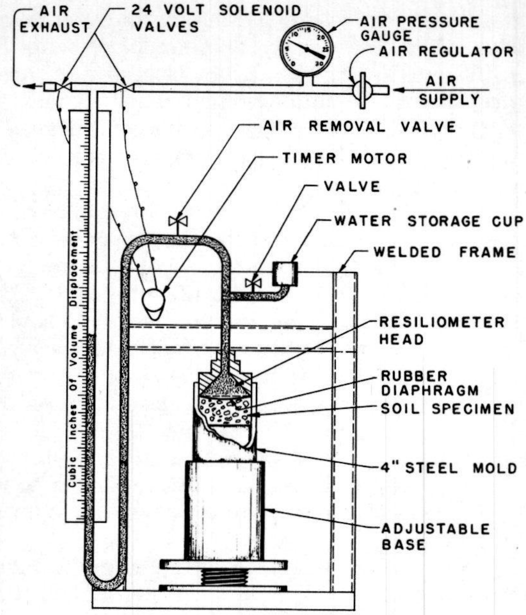


Figure 28. Resiliometer, schematic diagram.

As granular materials (such as clean sands, gravels, or crusher-run bases) characteristically show very-low values in the resiliometer, it is beginning to appear that there may be a closer correlation and parallelism between results in the stabilometer and measurements in the resiliometer than was first expected. Before resiliometer results were available, the idea was entertained that, when the soil pores were filled with water, the combination would be virtually incompressible and, consequently, the resilience ought to be very low. Actual tests, however, have shown that if the soils do display any appreciable

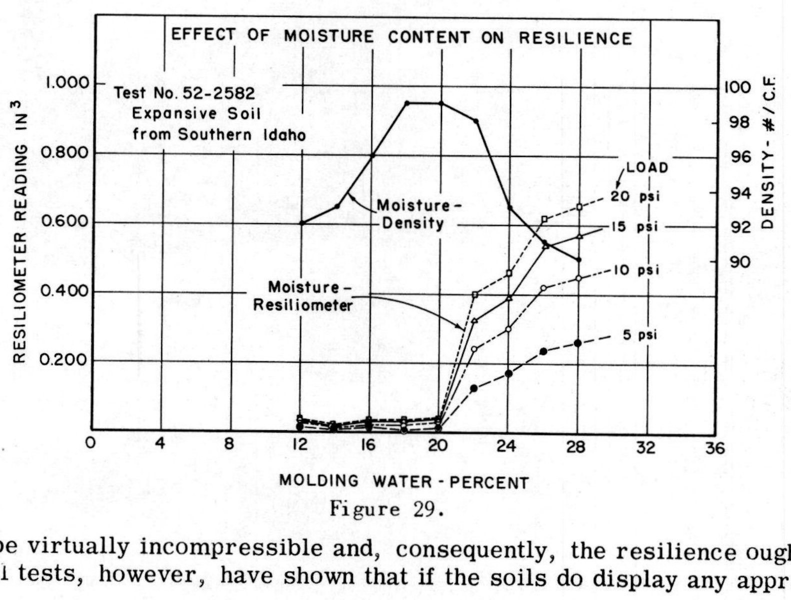


Figure 29.

resilience the range of movement increases with increasing moisture content beyond the point where the voids are first filled with water. Obviously, any granular mass cannot contain more water than the void space will accommodate. But while this void space is comparatively stable and fixed for a clean sand or gravel, such is not the case in fine-grained soils of the expansive type. Here the capacity for moisture will increase markedly as the soil expands. On such soils the internal resistance (R-value) will diminish,

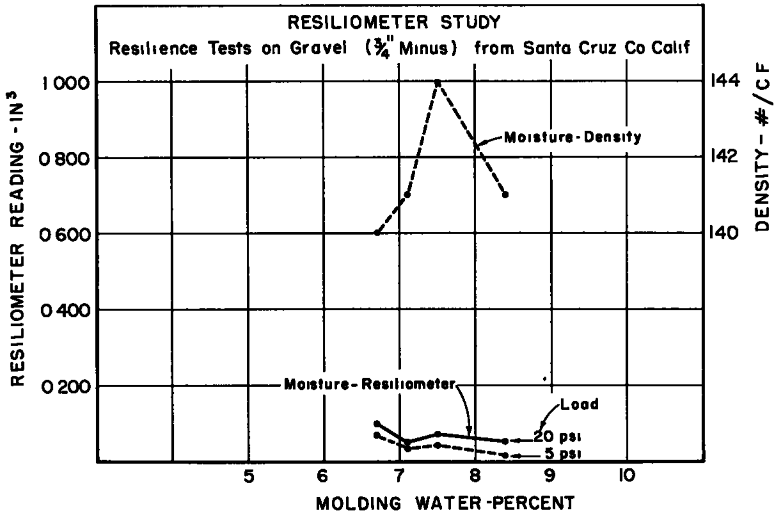


Figure 30.

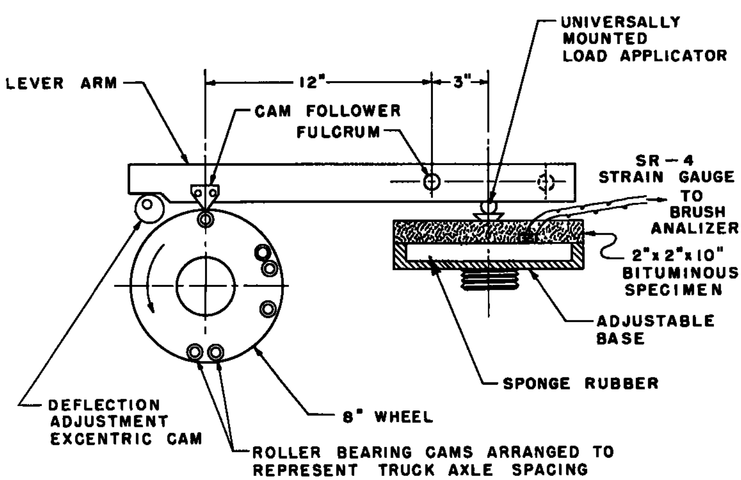


Figure 31. Bituminous pavement fatigue tester.

but it appears that the springiness or resilience will increase.

While the foregoing trends seem fairly evident, a great deal of work is yet to be carried out before the resiliometer becomes a proven device for the routine testing and evaluation of the resilient properties of soils.

FATIGUE RESISTANCE OF ASPHALTIC PAVEMENT

A second evaluation which must be made in the process of rational design concerns the ability of various types of pavement to withstand continued bending and flexing under the repeated action of traffic. How flexible is a "flexible pavement"? This character-

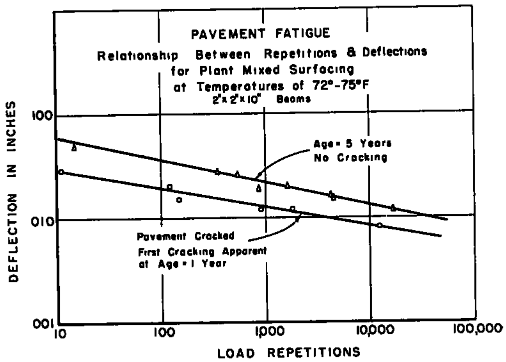


Figure 32.

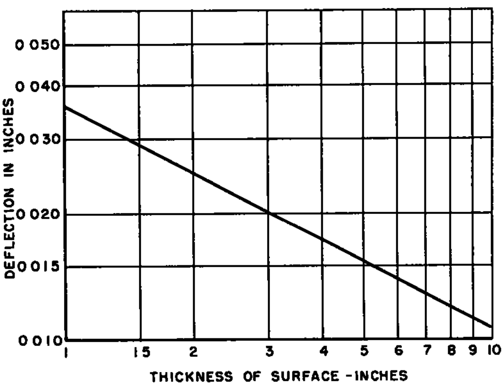


Figure 33. Permissible deflection under 10 million equivalent wheel loads.

istic will be more difficult to evaluate individually by test of pavement samples performed in advance of actual construction. In the first place, it will be difficult, if not next to impossible, to manufacture laboratory specimens that will have all of the properties of aged asphalt pavements that have been under traffic for some years. At the present time it seems questionable whether satisfactory and truly representative specimens can be formed in the laboratory for the purpose of measuring pavement flexibility. Fortunately, it does not appear that such a procedure will be absolutely necessary, as it should be possible to establish characteristic limiting values typical of pavements which have been in use for several years.

It is well known that asphalts tend to harden with the passage of time; therefore, asphalt pavements are undergoing constant change in their properties because of oxidation, loss of volatiles, polymerization and increasing density under the action of traffic. It seems that any evaluation of the ability of an asphalt pavement to withstand fatigue failures must be based on observations of actual performance on the road. Characteristic safe values can be set up for design purposes as is common practice for all structural materials. However, it should be possible to confirm evi-

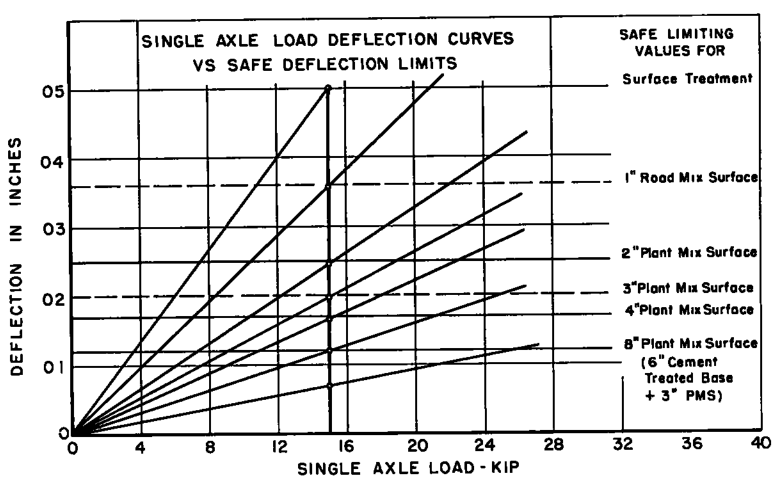


Figure 34.

dences of performance by the use of beams or test specimens sawed from actual pavements.

Reports of work at the University of Illinois on the fatigue of partland-cement concrete have indicated that a sufficient number of repetitions of a load that equals only 50 percent of the modulus of rupture value will ultimately cause failure (5). The modulus of rupture value is not easily or accurately determinable on a ductile, yielding material such as a specimen of asphalt pavement. However, studies have been made on the fatigue

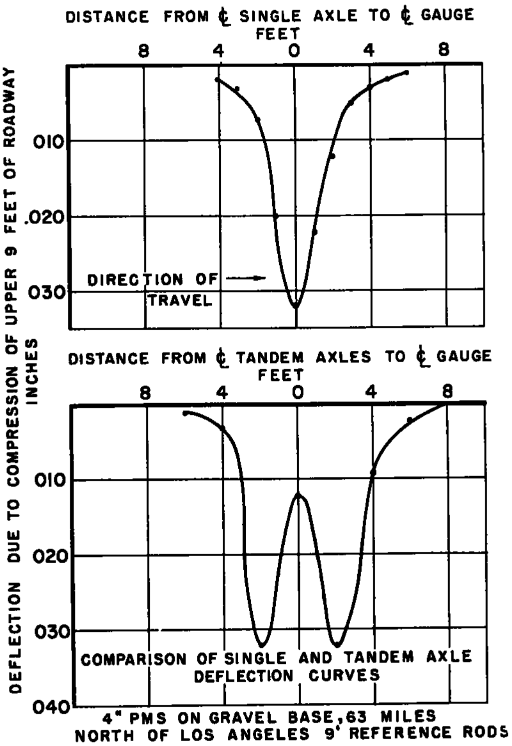


Figure 35.

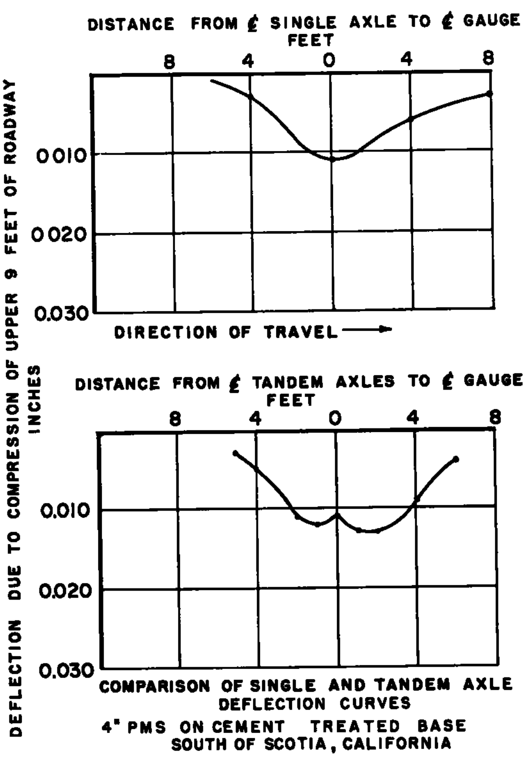


Figure 36.

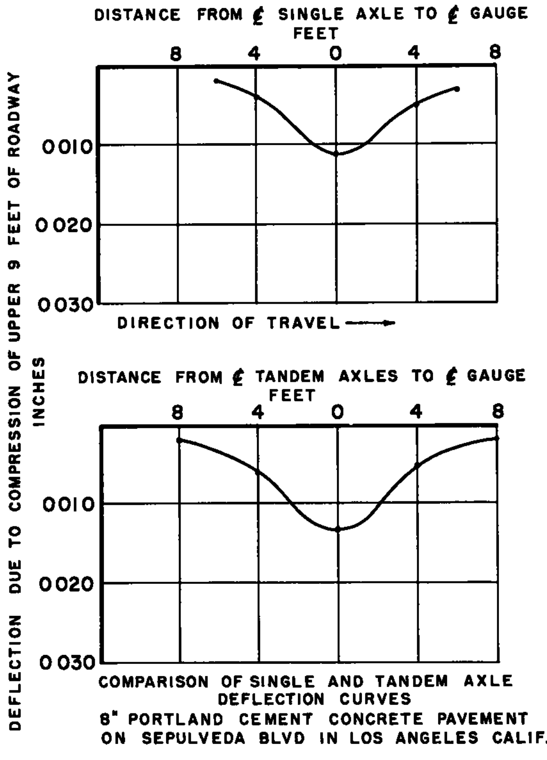


Figure 37.

resistance of small asphaltic pavement beams in the University of Washington under the direction of R. G. Hennes (6). Also, recent reports from Sweden indicate an interesting relationship between tensile

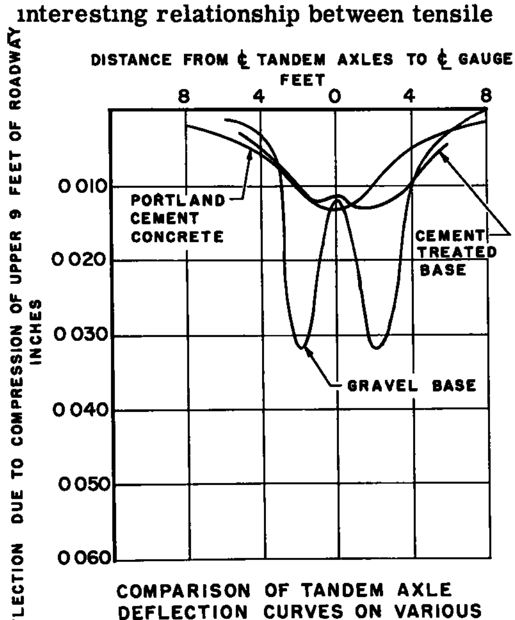


Figure 38.

strength and temperature (7).

Studies are now under way in this laboratory with a newly constructed device for measuring the fatigue resistance of typical asphalt pavements (Figure 31). Only a few results are available at the time of writing this paper, and the device for measuring fatigue or flexibility of bituminous pavements will undoubtedly undergo some changes and improvements, chief of which will be means for maintaining accurate temperature control. However, the initial trial results are interesting, and Figure 32 illustrates the results obtained on small beams of asphaltic pavement cut from slabs taken from a road surface. These preliminary results are unexpectedly uniform and show a definite relationship between the magnitude of deflection and the number of repetitions required to produce failure.

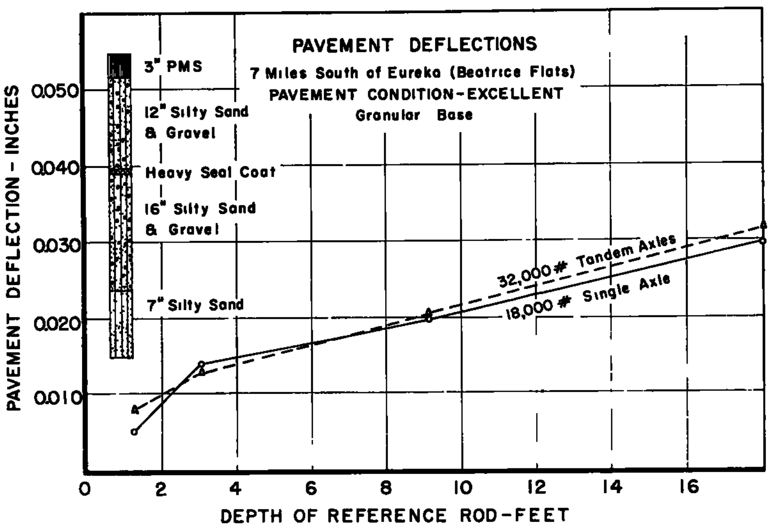


Figure 39.

It will take some time to establish the relationship between deflections of these small beams and the deflection of a pavement slab under heavy wheel loads, but there is little room for doubt that such a relationship does exist. Curve A in Figure 32 represents an uncracked pavement. Curve B represents beams from a cracked pavement. It is evident that the quality of the pavement is one of the variables.

In connection with the work of the WASHO Road Test, William Carey, of the Highway Research Board, experimented with a device to apply sudden rapid loads to beams of an asphaltic pavement. He suggested that we might do some work along the same lines and also suggested the use of a soniscope. Soniscope tests have been performed in our laboratory with some interesting results tending to confirm the work reported from Sweden (7); however, we believed that it would be necessary to subject pavement specimens to repeated loading in order to simulate actual road conditions. Hence, the device for measuring fatigue susceptibility in asphalt pavements was constructed with a cam arrangement operating at speeds to simulate the sequence of wheels on a multiple-axle truck.

In the preliminary trials, small beams 2 by 2 by 10 inches cut from asphaltic pavements have failed by cracking after being deflected 0.008 inch repeated 12,000 times at a temperature of 75 F. to 78 F. While this magnitude of deflection on a short beam can not, as yet, be compared directly to the deflections measured on the roadway (Figure 13), there can be little doubt that such pavements would fail after fewer repetitions when temperatures are lowered to the range typical of winter conditions throughout most of the United States.

RATIONALIZATION OF THE DATA

In the light of the foregoing, it may be visualized that, in addition to the design chart suggested in 1948 (1), a second design process will need to be established to provide a sufficient depth or strength of pavement which will reduce the deflection to a value which the pavement can successfully tolerate throughout its entire economic life or to find means for constructing a flexible pavement that will not be damaged by the magnitude of bending stresses involved.

It appears that we can now make a start in suggesting tentative values for a safe scale of permissible deflections for current pavement types. It is obvious, of course, that the overall flexibility of any engineering material will vary with the thickness of the slab or beam, other things being equal.

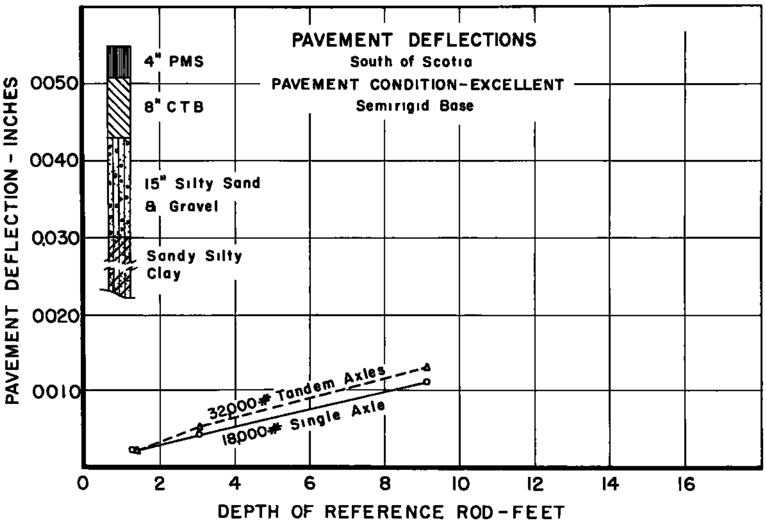


Figure 40.

Observations of these deflections and pavement performance seem to justify the suspicion that any superior flexibility of present day asphaltic pavements may be due largely to their generally thinner sections and, of course, varies with the amount of asphalt and age. The data seem to raise the question: Are present day asphalt pavements ultimately any more flexible than concrete pavements if constructed to the same thickness and if compared at low temperatures? Subject to many exceptions and individual variations, however, Table 1 appears to be a reasonable approximation of values for safe maximum deflections for several types of pavement and base construction.

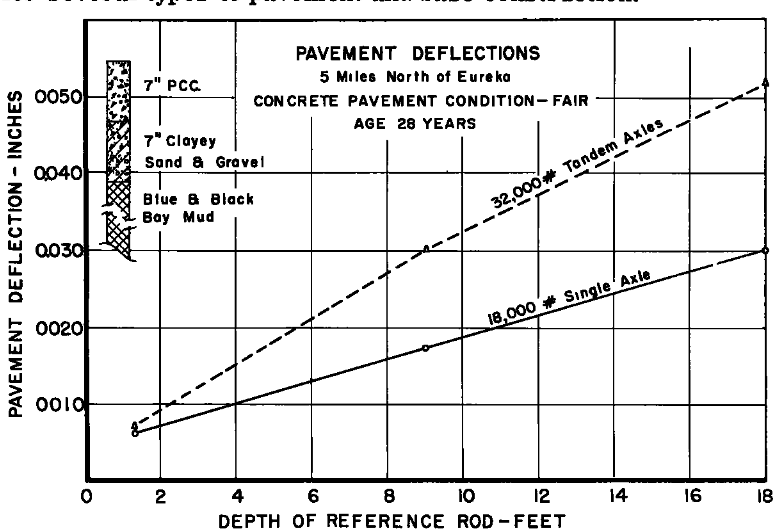


Figure 41.

TABLE 1

Thickness of Pavement	Type of Pavement	Max. Permissible Deflection for Design Purposes (Tentative)
8-in.	Portland Cement Concrete	0.012-in.
6-in.	Cement Treated Base (Surfaced with Bituminous Pavement)	0.012-in.
4-in.	Asphalt Concrete	0.017-in.
3-in.	Plant Mix on Gravel Base	0.020-in.
2-in.	Plant Mix on Gravel Base	0.025-in.
1-in.	Road Mix on Gravel Base	0.036-in.
½-in.	Surface Treatment	0.050-in.

(Bear in mind that the thickness of pavement indicated may or may not be adequate to satisfy the demands of problems one and two as outlined previously.)

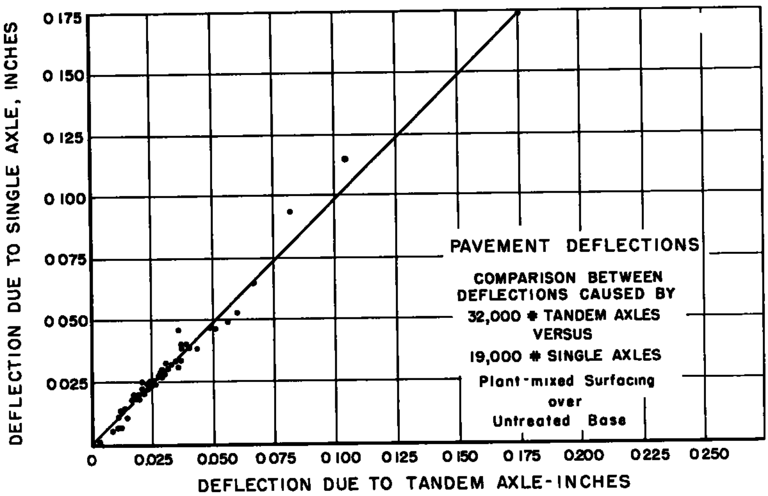


Figure 42.

The tentative deflection values of Table 1 are shown as a curve (Figure 34). The curve intended to indicate the safe limits of deflection under some millions of repetitions by heavy wheel loads. It is here assumed that a mean value would range somewhere in the neighborhood of 15,000-lb. single-axle loads. Many instances are known, of course, where the heavy traffic is almost entirely represented by trucks hauling logs, gravel, or similar commodities where all axle loads will be near to (or frequently exceed) the legal limits. Figure 33 shows a suggested straight-line relationship on a logarithmic grid between pavement thickness and permissible deflection.

In view of the fact that the majority of the deflections indicate a linear relationship between deflection and magnitude of the axle or wheel loads, a chart may be constructed, (Figure 34) showing the relative deflections which would be developed under a range of loads, the lines being drawn through points above the 15,000-lb. axle load corresponding to the various maximum deflections suggested in Table 1 and Figure 33.

A relationship suggested by W. N. Carey and A. C. Benkelman may prove to be more-consistently significant than simple deflection measurements alone. This relationship is expressed as

$$b = \frac{d}{a}$$

where b = bending index
d = deflection in inches

a = one half the axis of the load deflection area; that is, the distance in inches from the center of tire contact to the edge of the deflected area.

TANDEM AXLES VERSUS SINGLE AXLES

Among interesting facts brought to light by these field measurements of deflections is the evidence of a variable, but apparently orderly, relationship between the deflections resulting from single-axle loads compared to those caused by loads placed in the close proximity that occurs with tandem axles.

In the majority of pavements studied, the deflection measurements have been recorded for both single-axle and tendem-axle loads. Figure 35 shows the rapid reversal of pavement bending under a 31,000-lb. tandem axle on a section of badly cracked asphaltic pavement illustrated in Figure 6 and, for comparison, the pattern registered by a single-axle 18,000-lb. load. Figure 36 shows the deflections under both a single axle and tandem axles on the excellent pavement supported by a cement-treated base shown previously by Figure 17. Figure 37 illustrates the deflections of a concrete pavement under a single-axle load and the deflections of the same slab under tandem axles.

For bituminous pavements on gravel bases the foregoing indicates that when two axle loads are closely spaced, as in the case of tandem axles, then each trip of a truck produces a repetition of load for each axle regardless of spacing. For concrete pavement, on the other hand, there is little or no rebound between such closely spaced axles; therefore, a tandem axle should be counted as one axle load for purposes of summarizing the effects of traffic in equivalent wheel load (EWL) computations (see Figure 38 for comparisons, for instance).

The differences between the effects of single-axle and tandem-axle loads are further evident when the total amount of pavement deflection is compared. Figure 39 illustrates that an 18,000-lb. single axle and a 32,000-lb. tandem axle produce almost exactly the same deflections for depths up to 9 feet. This is the same pavement referred to by Figure 16.

This same close correspondence has been noted on most of the pavements consisting of a bituminous surface over a granular base. Figure 40 shows that where a cement-treated base exists there is a small, but consistent difference, the semirigid base showing greater deflection under a tandem representing two 16,000-lb. axle loads than for a single 18,000-lb. Finally, Figure 41 shows the marked difference where a concrete pavement is involved. Here the deflection under a 32,000-lb. tandem axle is much greater than for the 18,000-lb. single axle.

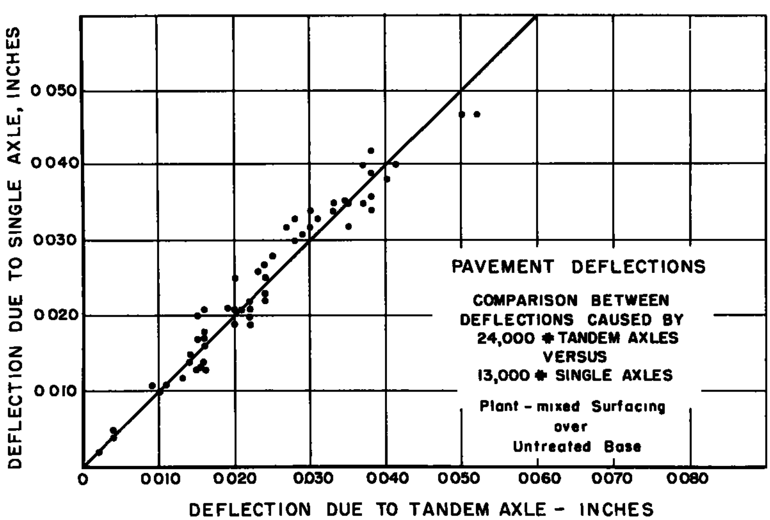


Figure 43.

Thus, it appears that the deflections of bituminous pavements over a gravel base subjected to a 32,000-lb. load on tandem axles show an average value almost identical to that produced by a 19,000-lb. single axle. Figure 42 shows the close and consistent relationship between all the available deflection values on bituminous pavements over gravel bases for both tandem and single axles at these loads. Similarly, Figure 43 shows that a 24,000-lb. tandem is equal to a 13,000 single axle. However, the relationship over a cement-treated base is indicated in Figure 44 as 32,000-lb. tandem equals 21,000-lb. single. For portland-cement concrete (Figure 45) 32,000-lb. tandem equals 24,000-lb. single.

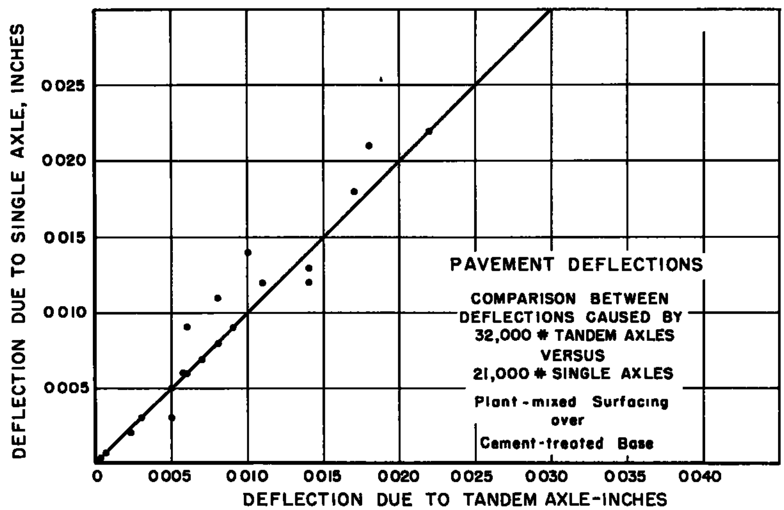


Figure 44.

This last relationship seems to be strikingly confirmed by an examination of the data for Road Test One-MD, where the destructive effect as indicated by the lineal feet of cracking produced by the 32,000-lb. tandem axles appears to be almost exactly equal to the amount that would be indicated by extrapolation for the same number of trips of a 24,000-lb. single-axle load.

Extrapolating the trend described above means that an analysis of strength requirements for a bridge deck should show still less difference between a load carried on single axles as compared to tandem axles. The relationship will vary with the length of the span, but according to the AASHO formula for a 32-foot span, a 32,000-lb. tandem-axle load should be equal to a 28,000-lb. single-axle load.

The interrelationship between load distribution and pavement strength is illustrated graphically by Figure 46. This chart indicates the "safe" deflections for several types of pavement and the relative deflection that would result for any condition from any change in load either single axle or tandem. The effect of slab strength is evident. Figure 46 is an attempt to rationalize the data where the deflections vary directly with load and indicates the orderly relationship between magnitude of load, axle spacing, pavement type, and deflections.

Mention was made earlier of the concept of stiffness and reference was made to work with the Shell vibration machine. In closing it should be mentioned that there appears to be some correlation between evaluations tentatively established by Nijboer and associates for pavements in Europe and the indications derived from deflection measurements in California. As outlined in the footnote, the concept of stiffness as used by Nijboer covers the total resistance of the pavement, base, and soil. This is also true of the deflection measurements as reported herein. Therefore, the relationship between stiffness and deflection may be expressed by the formula

$$S = \frac{L}{d}$$

Where S = the stiffness in kip per inch
d = the deflection in inches
L = the wheel load = $\frac{\text{axle load}}{2}$

Figure 47 shows the relationship between axle loads, deflections, and computed stiffness. As stated previously and indicated in Table 1, we have tentatively assigned limits for the deflection that pavements of various thickness and type will safely withstand over a period of years. These values, of course, are only tentative at this time, but it appears that the heavier pavements should be limited somewhere between 0.012 inch and 0.020 inch.

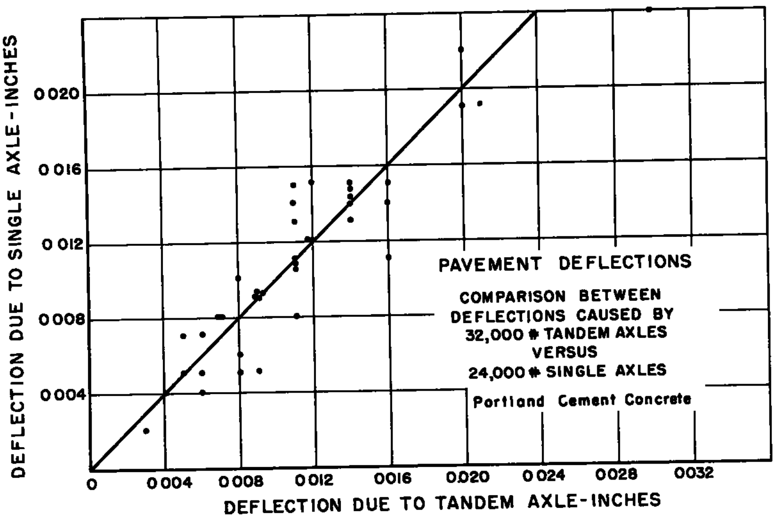


Figure 45.

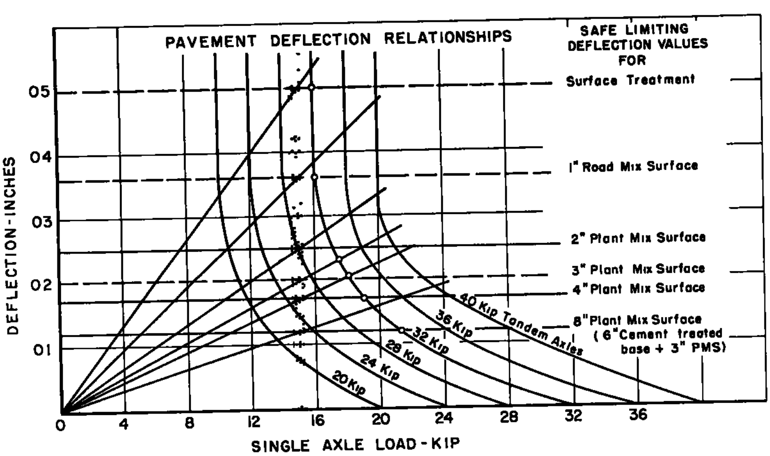


Figure 46.

Referring to Figure 47, this range of deflections is the equivalent of a stiffness factor in the approximate range of 400 to 600 kip per inch. For comparison, the limiting value of stiffness suggested by Nijboer and van der Poel (2) ranges from 570 to 1,140 kip per inch. This agreement is not too close, but the comparison is offered primarily to indicate that our present ideas of a limiting deflection are liberal rather than otherwise. If we accept Nijboer's conclusions, permissible deflections would range between 0.006 inch and 0.013 inch.

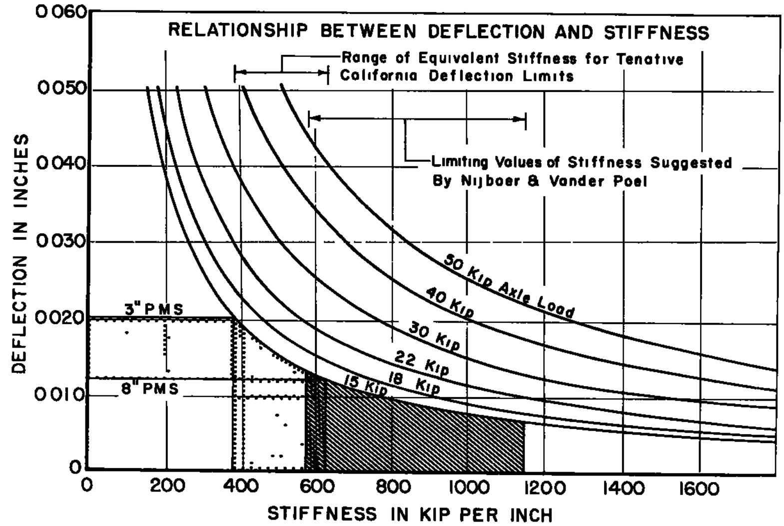


Figure 47.

CONCLUSIONS

In conclusion then it may be stated that there is an unusually close correlation between observations of cracking and fatigue type failures in bituminous pavements and the measured deflections which the pavement must undergo with each passing wheel load. These deflections appear to be associated with compression and rebound in the soil, and it is obvious that most pavements will withstand a few such deformations if not too often repeated. It may be said that a principal destructive force is the energy stored in the subgrade by each passing wheel.

It appears that the problem has three possible solutions: (1) provide a pavement[5] or wearing surface layer that is sufficiently flexible to accommodate repeated substantial vertical deflections without serious cracking; (2) decrease the magnitude of the vertical deflections to a tolerable limit by providing greater stiffness by means of greater depths of granular bases and subbases under the pavement; or (3) provide a pavement with a slab strength sufficient to sustain the forces induced by traffic which cause cracking.

With regard to Solution 1, above, the only method of utilizing flexibility, at present, is to use a thin bituminous surface treatment as there are no materials available today at reasonable cost to construct truly flexible surfaces of substantial thickness. Such thin "pavements" are, of course, vulnerable to other destructive effects of heavy traffic and to adverse weather conditions. Also, a thin surface is not able to provide the necessary strength or weight to carry loads over cohesionless sands or over plastic soils.

Solution 2 is in recognition of the fact that the magnitude of the deflections are related to the overall pavement structure thickness and that actual deflections can be reduced to an acceptable limit simply by increasing the thickness of a non-resilient base or subbase. At present, pavement and base thickness design procedures are predicated primarily on ability to carry loads over plastic soils (i. e., on resistance values and expansion pressures), which means providing sufficient cover thickness to support traffic over soils of low bearing or resistance value.

It appears that it will now be necessary to develop a second pavement structural design procedure based on resiliency factors and fatigue susceptibility in which soil resilience, magnitude of loads, load repetition and the stiffness of the pavement and base are all related in order to provide an adequate design. Both procedures will then have to be considered, and the thickest of the two pavement sections selected.

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[5]The term "pavement" as used herein, includes that portion of the overall pavement structure lying on the base and which is generally a mixture of aggregate and asphalt or portland cement.

It appears to be true, fortunately, that in many cases (perhaps in a majority of instances) low resistance values and high resilience characteristics go hand-in-hand; consequently, most pavements designed on strength factors are adequate from the standpoint of resiliency effects. However, there is evidence that this is not always the case, and there does exist an element of doubt which should not be allowed to exist if it can be eliminated.

Solution 3 calls for a pavement of considerable slab strength, such as heavy portland-cement-concrete slabs or the use of cement-treated bases in conjunction with a substantial thickness (4 inches minimum) of asphalt pavement. It has been observed, and substantiated to a large extent by the data included in this paper, that pavements or pavement structures of high slab strength need not be as great in overall thickness as sections utilizing lower slab strengths in order to perform satisfactorily in carrying traffic over resilient soils. However, for modern industrial traffic all sections must still be of substantial thickness.

It appears that the engineer is faced with the necessity of designing pavements of the various types described above in order to meet all three primary problems: (1) potential expansion, (2) plastic deformation, and (3) the resilience of the underlying materials. After designing comparable sections which will satisfy the above structural and physical requirements, it is then the engineer's responsibility to make an economic analysis to determine which one should be specified for a given location.

None of the foregoing promises to make life any simpler for those who must design highway or airport pavements.

ACKNOWLEDGMENT

The foregoing represents selected examples and extracts from data obtained in an extensive investigation which has been under way for over four years; so it is difficult to list by name all of those who have been involved and who have contributed time and effort to the various phases of this work.

Most of the studies have been carried on in the Pavement Section of this laboratory under the direction of Ernest Zube. The principal task of correlating and assembling the data, analyzing and studying the results has been under the direction of George Sherman, assisted by Robert Bridges. The cutting of cores, securing soil samples and installing gauge units were handled by Charles Clawson. James E. Barton and assistants were responsible for the General Electric travel gauge, electronic equipment, and recording of the deflections.

I should like to acknowledge the courtesy and assistance of the Shell Oil Company, which made available the Shell vibrator and the services of a skilled operator in order to compare "dynamic" deflections and other measurements without the necessity for constructing such an expensive unit.

The author has appreciated helpful discussions with W. N. Carey, Jr., of the Highway Research Board, and A. C. Benkelman, of the Bureau of Public Roads.

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Discussion

EARL C. SUTHERLAND, Bureau of Public Roads—The paper by Hveem deals principally with pavements of the flexible type but contains also some data and comments pertaining to rigid pavements. This discussion relates primarily to that part of the paper which discusses rigid pavements, but some data from tests on flexible pavements will be presented.

During the past 35 years a large amount of work both of a theoretical and an experimental character has been done in this country and in several European countries on the subject of the design of rigid pavements. A splendid summary of the researches on this subject, during this period, is contained in a publication (in English) by the Swedish Cement and Concrete Research Institute at the Royal Institute of Technology in Stockholm, 1949 (1).

The first comprehensive theoretical analysis of the stresses and deflections caused by loads acting on rigid pavements was made by Westergaard in the middle 1920's (2). In this investigation three cases of loading were investigated as follows: (1) corner, a wheel load acting close to a rectangular corner of a large panel of a slab; (2) interior, a wheel load acting at a considerable distance from the edges; and (3) edge, a wheel load acting at the edge but at a considerable distance from any corner.

In this study the influence of the size of the loaded area was investigated and full subgrade support was assumed for all cases of loading.

During the 1930's the Bureau of Public Roads carried out a series of experimental researches, known as the Arlington investigation, one phase of which was devoted to a study of the various aspects of the Westergaard analysis. The results of this investigation were published in 1943 (3) and showed that the stresses and deflections computed by the Westergaard formulas were in close agreement with the stresses and deflections determined by measurement for the interior and edge cases of loading, but were somewhat smaller than the critical measured values for the corner case of loading. The lack of agreement for the corner loading was due to the fact that the ends and corners of a pavement slab warp upward during the night and thus do not have the perfect subgrade support assumed by Westergaard.

During the Arlington tests a large number of tests with the corner loading were made at night when critical upward-warping conditions prevailed. The data obtained were used to develop an empirical modification in the Westergaard corner formula. The corrected formula is sometimes referred to today as the BPR or the Kelley corner formula (3, 4). These same data were used also in the development of both the Spangler and Pickett empirical corner formulas, the latter being that recommended by the Portland Cement Association (5).

The theoretical and experimental studies of the design of rigid pavements made up to this time are of significance with respect to the paper under discussion.

In Table 1 the author gives what are termed "Maximum Permissible Deflections for Design Purposes (Tentative)". The deflection value for concrete pavements, 0.012 inch, was determined for the interior case of loading and was apparently selected because the subgrade acts in a manner highly elastic, or resilient, as it is termed in this paper, at the interior.

Cracking in concrete pavements is, of course, related to the stresses and the data obtained in the earlier work referred to above shows that the deflections of a concrete pavement slab caused by loads at the interior and edges are of little significance as an indication of the stresses produced by the loads (2, 3). For example, for these two cases of loading the size of the bearing area has a negligible influence on the deflections but has an important influence on the stresses caused by loads. However, for the corner loading the deflections, under some conditions, are of significance in predicting stresses.

While deflection values are of little significance as a measure of the magnitude of the stresses that might be caused by loads acting on a concrete pavement, their magnitude is important with respect to the development of pumping and consolidation of the subgrade. The most-serious structural damage that results from pumping is a slab-end failure where transverse cracks develop at 6 to 8 feet from the slab end. This failure is not

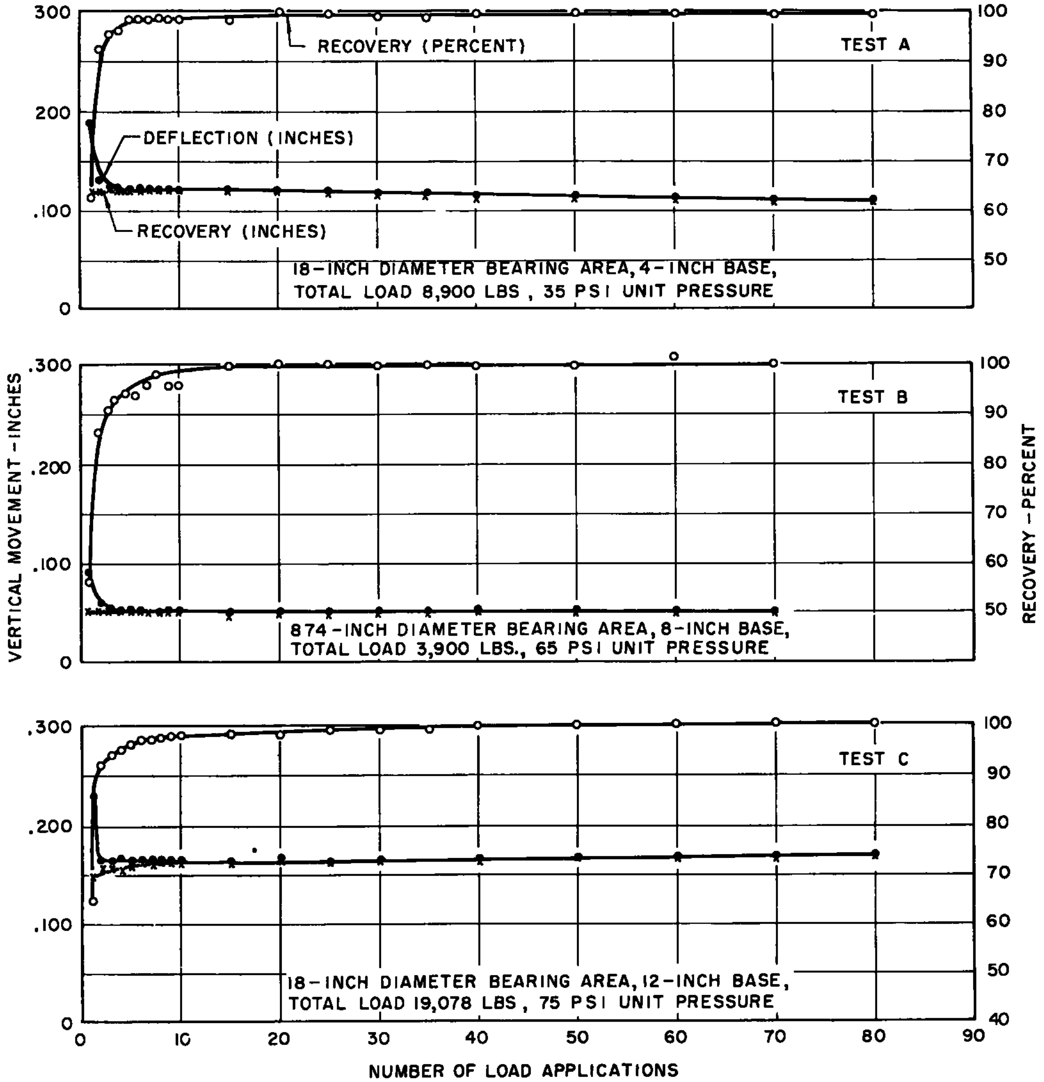

Figure A. Repetitional load tests on flexible pavement bases of different thicknesses, individual deflections and recoveries.

caused by any of the cases of loading analyzed by Westergaard, although in some respects it resembles the corner case of loading.

In the Arlington investigation (3) tests were made to study the elastic action of the subgrade under concrete pavements and also the magnitude of the deflections that develop under normal loading for the different cases analyzed by Westergaard. It was found that for the edge and interior cases of loading the subgrade acted elastically while at the corner (6) it did not. This is in agreement with the author's findings. It is probable that a reduction in the magnitude of the corner deflections would result in more nearly elastic action of the subgrade in the vicinity of the corner. However, if the pavement were to be designed with sufficient thickness so that the deflection at the corner will not exceed approximately 0.012 inch the result would be an unduly thick pavement.

The deflection data obtained in the Arlington tests were for static loads of a magnitude that would develop transverse bending stresses of half the modulus of rupture of the concrete. With this criterion for load magnitude it was found that the ranges in deflections for the different cases of loading on slabs 6, 7, 8, and 9 inches in thickness were about as follows: corner loading, 0.040 to 0.055 inch; interior loading, 0.008 to 0.012 inch; and edge loading, 0.018 to 0.020 inch.

Based on certain deflection data obtained in tests on concrete pavements (see Figure 45), the author makes the statement that a 24,000-lb. single-axle load is equivalent to a 32,000-lb. tandem-axle load. While not definitely so stated, these deflection data appear to have been obtained from tests at the interior of the pavements. It is stated that this selection of equivalency is supported by the crack data obtained in the Road Test One-MD investigation (7).

Except for a moderate amount of longitudinal cracking which occurred in Section 4, all of the cracking that developed in Road Test One-MD was in the vicinity of the slab ends and was caused by a slab-end loading, (called a corner loading in the report). This cracking appeared only after appreciable pumping had developed in the vicinity of the slab ends.

As pointed out earlier, the deflections caused by loads acting at the interior of a concrete pavement slab bear no direct relation to the magnitude of the stresses caused by the same loads. Furthermore, since cracking in the Maryland road test developed only after bad pumping had occurred, it is the writer's opinion that the amount of cracking should not be used as a criterion of load equivalency, particularly when applied to modern pavements where effective provisions have been made to control pumping.

In the report on Road Test One-MD, load-stress and load-deflection curves are pre-

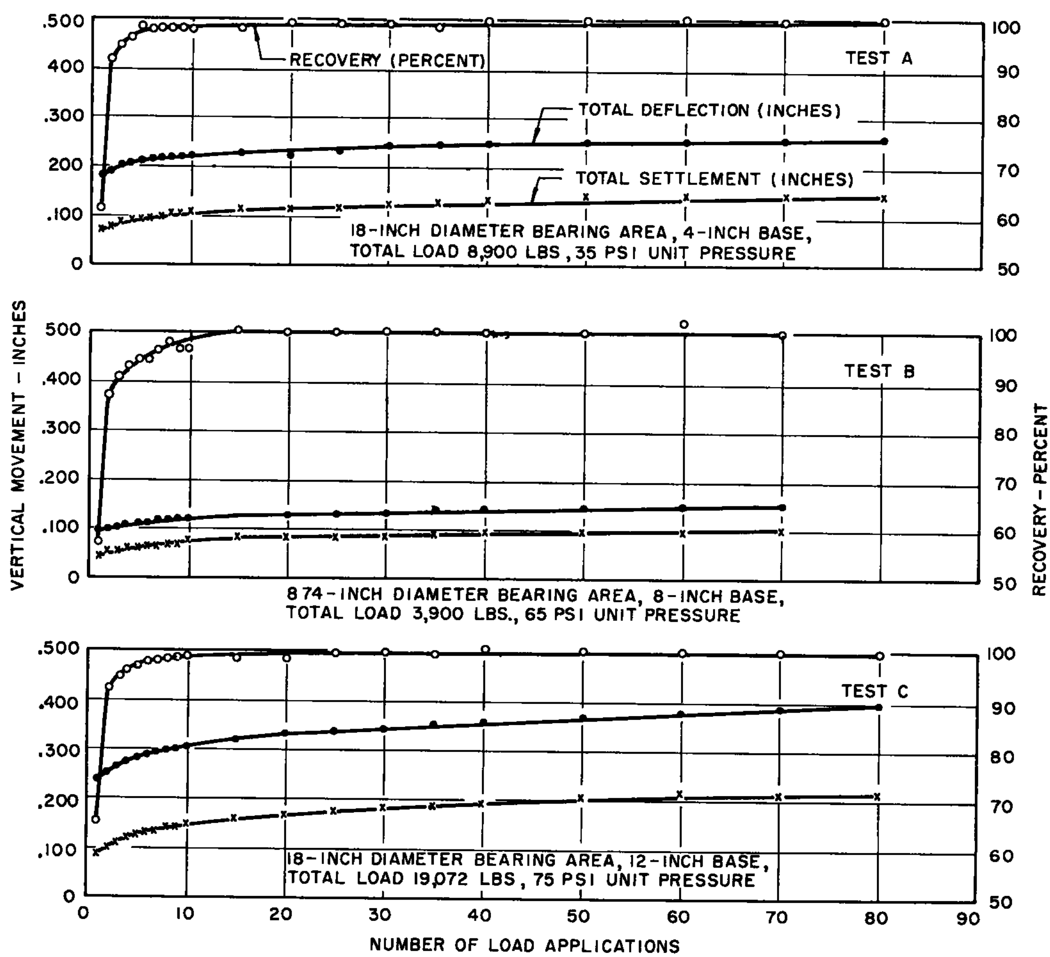

Figure B. Repetitional load tests on flexible pavement bases of different thicknesses, total deflections and total settlements.

sented for all of the cases of loading investigated for both the granular nonpumping soil and the pumped fine-grained soil. These curves are plotted in a manner such that equivalent single- and tandem-axle loads can be determined.

The interior loading is not critical and should not, therefore, be used for the selection of equivalent single- and tandem-axle loadings. As stated earlier, slab-end failures, caused by what was termed corner loading in the Road Test One-MD report, appear to be the most-serious type of structural damage to be found in concrete pavements today. For this type of loading on the granular nonpumping soil it is found, Figure 87 of the report, that at creep speed the 32,000-lb. tandem axle is equivalent to an 18,000-lb. single axle (7).

The author presents certain data pertaining to the "resilience" of flexible pavements and discusses this subject at some length. The Bureau of Public Roads has investigated this subject in a number of tests made over a period of years, and samples of the data obtained may be of interest in this discussion. However, the term "elastic action" rather than resilience has been used by the bureau. The object of the tests was to determine whether, under applied loads, the elastic action of the various components of a flexible pavement could be used as a criterion for determining the load carrying capacity of the structure. Flexible pavements carrying a large amount of traffic must act nearly elastically if rutting is to be avoided and the pavement is to remain smooth.

Figures A and B show samples of data obtained in tests made in 1946. The subgrade under the pavement on which these tests were made was of the A-6 group and on it there were three thicknesses of gravel base, 4, 8, and 12 inches, respectively.

As indicated in the figures, 70 to 80 static loads were applied on top of the base in each test. The loads were maintained for 2 minutes, and 2 minutes were allowed to elapse after the removal of each load before applying the next. As indicated on the graphs, different diameter bearing areas and different unit pressures were used.

In Figure A the deflections and recoveries are plotted with respect to a base measurement made immediately before the application of the respective loads, while in Figure B they are plotted with respect to a base measurement made at the beginning of the load test series. Thus, Figure A shows the deflection and recovery for each load of a series, while Figure B shows the total deflections and settlements caused by the series of loads. The same data were used in constructing both graphs.

The tests included in these two figures were selected as being representative of those in which the pavement appeared to act nearly elastically. It will be noted that 100-percent recovery was indicated for all three tests after a considerable number of loads had been applied.

It may be observed in Figure A that there is a marked reduction in the deflections and an increase in the recoveries for the first 10 to 15 load applications, after which they remain approximately constant and equal.

The manner of plotting in Figure B is more sensitive than that used in Figure A, as it shows a progressive increase in the total deflections and total settlement, even though essentially complete elasticity is indicated on the basis of individual load applications. From Figure B it must be concluded that completely elastic action was not attained in any of the tests.

Figure C shows data from tests in which the loads caused definite failure of the pavement. The data of this figure were plotted in the same manner as in Figure A.

The data of Figures A and B show that, to determine when a pavement is acting in an essentially elastic manner, it is necessary to measure the deflections and settlements for a large number of repeated loads. If the deflections are to be measured at the level of only one component of the pavement structure it would seem most logical to measure the movement at the top of the base as was done in these tests.

If a pavement is to continue to act elastically under repeated loadings, it is necessary that the magnitude of the deflections be sufficiently small that the structure of the various supporting components will not be broken down. The author presents certain data on this point with respect to the pavement surface. It would appear to be desirable to so design the pavement surface as to enable it to withstand a greater degree of flexing, if this could be accomplished without sacrificing stability. This would make it possible to utilize the supporting power of the subgrade to a greater degree and might lead to

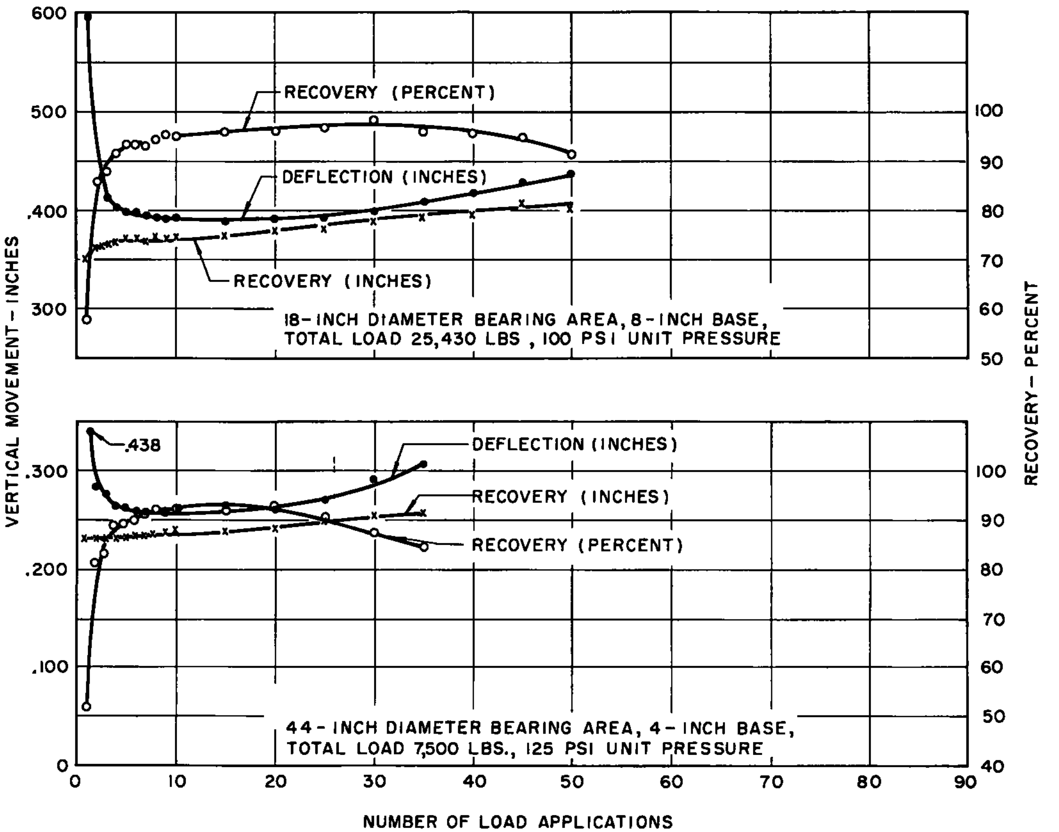

Figure C. Repetitional load tests on flexible pavement bases of different thicknesses, individual deflections and recoveries.

more-economical designs.

Investigations of flexible pavements in the past have been greatly handicapped by a lack of a dependable criterion for evaluating their load-carrying capacity. Elastic action might prove to be a good criterion for high-type flexible pavements, such as are used on primary highways. This subject should be pursued further for both repeated static and repeated dynamic loading.

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W H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska—Hveem has presented some interesting and provocative data. I wish to make the following comments.

Deflection or elastic deformation has been recognized as a factor in the designing of pavement thickness. For instance, the designers of concrete pavements select thickness on the basis of the subgrade modulus of reaction which involves the consideration of deflection. Furthermore, some designers of flexible pavements select thickness on the basis of deformation obtained by the use of plates. This deformation includes deflection.

Hveem has found that flexible pavements topped with a 2-inch bituminous surface are about twice as flexible as concrete pavements. This relationship seems to be consistent with expectations. However, the magnitude of the deflections which cause failure seem to be much lower than expected. For this reason it would be advisable to make deflection measurements over a wide area and on a wide variety of pavements before we draw definite conclusions.

Incidentally, the field deflection measurements could be made easily with the Benkelman beam. The use of this device would eliminate the possibility of errors due to the location of the reference point in the Hveem method. Hveem's data show that at any one location the deflection increases as the reference point is moved downward to depths of 18 feet and more.

The increase in deflection with the lowering of the reference point indicates that the effect of loaded surface areas extends to depths equivalent to several times the diameter of the loaded areas. This is significant to those who used plates for the evaluation of soils for foundation purposes. One of the principal objections to the use of plates for this purpose has been that the effect does not go far enough to engage the deeper soils.

Hveem reports that, in the testing of a plastic soil by the resiliometer, compressibility increased as water content was increased above the optimum. This result is at variance with basic fundamental principles. I hope he will determine whether the compressibility is due to entrained gases or the soil particles themselves. No doubt it can be assumed that water is incompressible.

I believe it can be shown that the deflection of a layered system is due principally to the soils beneath the base and that the deflection is usually due to the compressibility of entrained gases. Furthermore, the amount of the entrained gases is variable. Based on these assumptions, how can Hveem's resiliometer results on a small sample of soil in the laboratory be used to calculate deflections in the field?

STUART WILLIAMS, Highway Physical Research Engineer, Bureau of Public Roads and A. W. MANER, Assistant Testing Engineer, Virginia Department of Highways—Hveem is to be commended for his progressive thinking and action in connection with the problems of structural design of pavements. The work conducted in California under his direction serves to emphasize the great importance of the elasticity or resilience of the subgrade soil in the problem. It also points to the fact that information of considerable value can be obtained from deflection studies of flexible pavements in service.

In this connection, the results of a load-deflection study conducted cooperatively by the Virginia Department of Highways and the Bureau of Public Roads on a flexible pavement of modern design are of interest. The pavement, 5 miles in length, is located on State Route 7, east of Leesburg, Virginia. The old road on this location had a poor performance record which was attributed largely to the type of subgrade soil existent over much of its length. Accordingly, it was decided to rebuild rather than to strengthen the existing pavement.

The new pavement was designed by means of the Virginia method which is based on the CBR test with certain modifications[1]. Location is generally along the old route, but numerous changes in both horizontal and vertical alinement were made. Construction was completed in 1951.

A typical transverse section is shown in Figure A. Modified CBR values of a number of subgrade soil samples, tested in the laboratory, ranged from 1.5 to 17.0. Borrow material having a CBR of about 8 was readily available near the job and was used for

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[1] "Designing Flexible Pavements (Virginia)" by D. D. Woodson. Research Report 16-B, Highway Research Board, 1954.

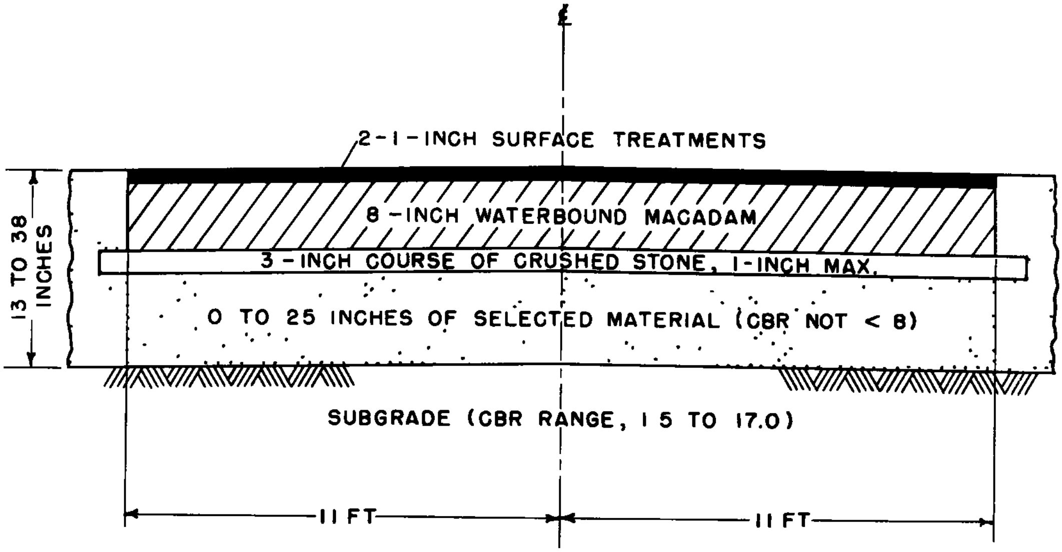

Figure A. Typical transverse section showing components of the overall pavement structure.

subbase in thicknesses of 0, 6, 12, 22 or 25 inches. The base course, uniform throughout the length of the project, consists of an 8-inch thickness of waterbound macadam constructed in two 4-inch courses. Underlying the base is a 3-inch course of crushed

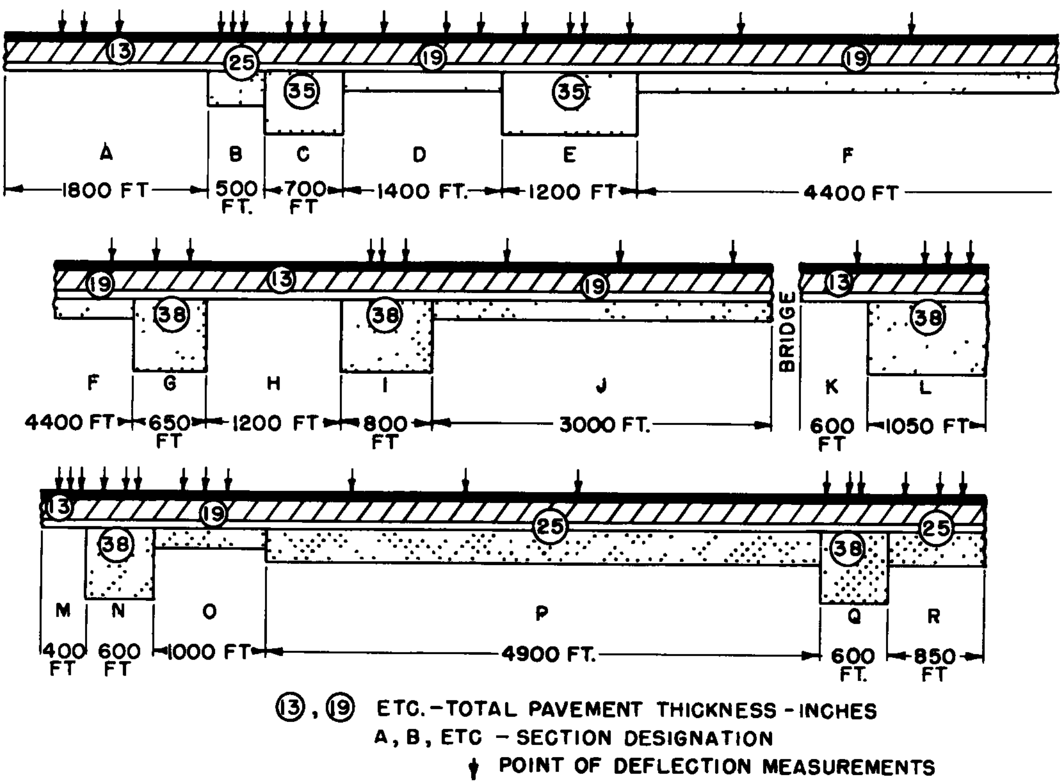

Figure B. Longitudinal section showing locations where deflection measurements were made.

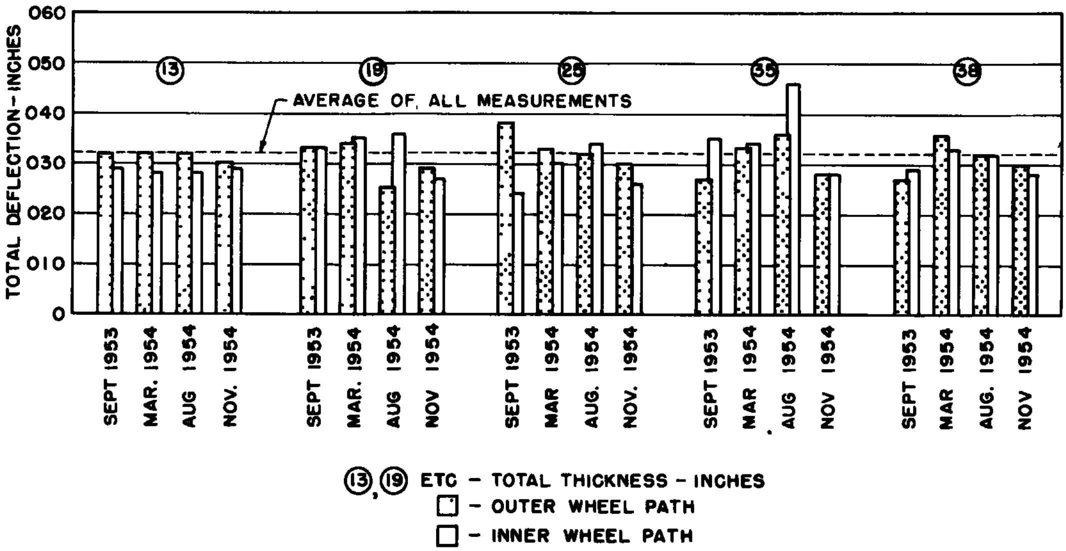

Figure C. Deflections obtained at different times of the year on the various pavement thicknesses.

stone, 1-inch maximum size. A surface treatment or armor coat about an inch thick was laid soon after completion of the base course. This was reinforced by a second similar treatment about 2 years later. Subgrade CBR test values, the corresponding five pavement design thicknesses constructed and the total length of pavement of each thickness are listed in Table A.

TABLE A

CBR test value	Total pavement thickness	Total length of pavement
	Inches	Feet
1.5	38	3,700
1.8	35	3,700
3.7	25	6,300
5.5 - 6.7	19	9,800
10.0 - 17.0	13	1,800

Additional information regarding the length, thickness and location of the individual pavement sections is shown graphically in Figure B. The sections, lettered A to R, range in length from 400 to 4,900 feet. Also shown in Figure B are the points at which deflection measurements were made. In general, measurements were obtained at three arbitrarily selected points in each section.

This investigation was conducted to: (1) obtain an indication of the deflection of the pavement as a whole; (2) develop information concerning the relative strength of the five different overall pavement thicknesses; (3) determine the uniformity in strength within each section of pavement having the same overall thickness; (4) compare the indicated strength of the pavement in the outer wheel path (edge of pavement) with that of the inner wheel path (interior of pavement); (5) study the elastic action of the pavement, i. e., to determine whether the application of the test load produces any permanent movement; and (6) determine the effect of seasonal changes on deflection.

The load-deflection tests were made in the fall of 1953 and in the spring, summer, and late fall of 1954. One day was required to complete the tests for each of these periods. A heavy duty two-axle truck equipped with 11.00-by-20 dual tires was used to apply a 9,000-lb. wheel load to the pavement at creep speed (1 to 3 mph.). The truck traveled in the eastbound lane with the centers of the rear wheels approximately 1.5 and 7.5 feet from the edge of the pavement. The lines of travel of the outer and inner wheels are referred to as the outer and inner wheel paths.

Pavement deflections were measured with the Benkelman beam deflection indicator which was developed for the WASHO Road Test in Idaho[2]. By means of this device a comparatively large number of load-deflection and recovery measurements were obtained in a relatively short period of time. At each location a single measurement was

[2] "The WASHO Road Test", Special Report 18. Highway Research Board, 1954.

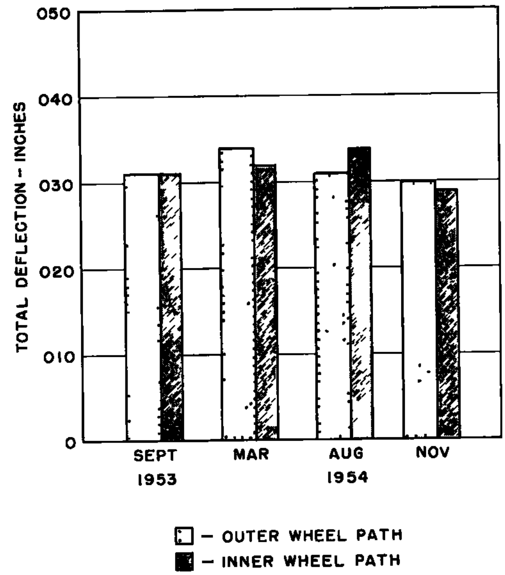

Figure D. Comparison of pavement deflections obtained at different times of the year. Average of all measurements for each test series.

made between the rear dual tires in each wheel path as the vehicle moved forward at creep speed.

From the structural viewpoint the pavement has performed well since it was rebuilt. During the first 2 years spot patching of a number of small areas of distress in the temporary surface treatment was required. It is believed, however, that this distress was due to raveling of the temporary surface treatment, rather than to structural instability of the pavement. At two of the deflection-test locations, both of which are on the poorer subgrade, some general cracking of the temporary surface, particularly near the pavement edge, had occurred. In order to augment the wearing course and to improve the smoothness of the pavement, a second surface treatment was applied over the entire project in the fall of 1953.

Traffic on this pavement in 1954 consisted of about 3,200 passenger cars, 125 tractor-trailer combinations and 525 light to medium-weight trucks per day.

Although this study should not be considered a comprehensive one and, as has been mentioned previously, consumed a minimum of time, several interesting findings have resulted. For any one test period the individual deflection values vary considerably between sections of the same overall thickness (about 0.025 inch) and even between points in the same section (about 0.015 inch).

The permanent vertical movement caused by the application of the one load of each of these tests ranges generally between plus and minus 0.002 inch. In over a third of the total tests made, however, the deflection and recovery are equal.

The averages of all measurements in the two-wheel paths for each of the overall pavement thicknesses are shown graphically in Figure C for the four seasonal test series. The individual bars represent averages of from 3 to 14 readings, the majority being an average of 6 or more. The following general comments may be made concerning these average values:

1. The deflection of the entire pavement for all test periods is 0.032 inch.
2. With the exception of a few erratic values, the deflection does not vary appreciably with change in pavement thickness. Values range from about 0.025 to 0.035 inch.
3. Comparatively little effect of seasonal changes is indicated, except that for all pavement thicknesses late fall deflection values (November) are somewhat smaller than those of the remaining test periods.
4. The difference between the deflections of the outer and inner wheel paths shows no consistent trend except for the 13-inch-thick pavement, where those of the outer wheel path are somewhat the greater.

In Figure D the average deflections for each period of testing are shown, each bar representing the average of all measurements in each wheel path. On the basis of these values, the largest deflections were obtained in March, although they are only slightly greater than those of the other periods. The minimum deflections occurred in November. It is apparent that seasonal changes had only a small effect on pavement deflections. It should be pointed out, however, that the winter of 1953-54 was comparatively mild and the precipitation during the period of this study was below normal. The deflections in the outer wheel path for the March and November tests are somewhat greater than those of the inner wheel path. However, this relation is reversed for the August tests, while in September the deflections in the two wheel paths are equal.

SUMMARY

The average of all the deflection measurements made on the Leesburg pavement is 0. 032 inch, with comparatively few individual measurements greater than 0. 040 inch. Performance under moderate traffic has been excellent; therefore, the pavement as constructed might be considered adequate. According to the data presented by Hveem, the safe deflection limits for an 18, 000-lb. single-axle load would range from 0. 030 inch for a 2-inch plant-mix surface to 0. 043 inch for a 1-inch road-mix surface.

Based on both visual inspection and average deflection measurements, there is little difference in the behavior of the sections of different overall thickness. In view of the fact that the subgrade consists of soils having a great range in load-bearing values, this indicates that a rather uniform strength pavement was obtained by the design method used.

It was found that the individual deflections as measured at the different locations in sections of uniform thickness varied to a considerable extent. Consequently, for a study of this nature a great number of measurements should be made so that local inconsistencies will have a minimum effect on the results. With the deflection measuring device used in this investigation, this is possible.

The pavement appears to react to load in a nearly elastic manner. At some locations the "permanent" displacement caused by the test load was slightly downward and in others slightly upward, but in over a third of the tests the recovery was equal to the deflection.

Seasonal changes caused little effect on pavement deflections. It might be expected that deflection values would decrease from a maximum in March to a minimum in November. Minimum deflections were measured in November, although they were only slightly less than those in March. The maximum movements occurred in the summer. Possibly the somewhat abnormal climatic conditions existing for the period of the study contributed to this result.

The assistance of A. W. Furgiuele, Materials Engineer, Culpepper District, Virginia Department of Highways, in the conduct of the deflection tests is gratefully appreciated.

F. N. HVEEM, Closure—Sutherland gives an outline of some of the investigations and theoretical analyses relating to stresses and deflections of rigid pavements. He points out that the Arlington investigation conducted by the Bureau of Public Roads failed to confirm the Westergaard formulas for computed stresses and deflections at the corners and ends of slabs.

An extensive field investigation of concrete pavements in California conducted some ten years ago partially reported in ACI "Proceedings" for 1951 Vol. 47, p. 797 supports Sutherland's statement about warping slabs. But in passing, I might offer the opinion that undue emphasis has been placed on the fact that the corners show the greatest amount of warping. In the hours between 5 A. M. and 7 A. M. the greatest curling is generally developed, the entire ends of each slab being lifted from the subgrade for a distance ranging from five to seven feet from the end of the slab, (see Figure 19 of ACI paper and see Figures A and B of this closure).

Repeated flexing of this "cantilever portion" of the slab accounts for the transverse crack which Sutherland mentions. These cracks develop along an irregular line about 6 to 8 feet from the slab end. Sutherland is correct in stating that this type of failure is not explained by the Westergaard analysis, but it is also true that this type of transverse failure rarely develops unless the subgrade soil is washed out or eroded away by the pumping action of the "curled up" free end of the slab. While slabs curl and "pump" on our cement treated subgrades they do not show the transverse crack and faulting is negligible.

In the measurement of deflections on California pavements we were not primarily interested in measuring the vertical movement at the slab ends as the amount of this movement varies greatly throughout the day, as shown by Figures A and B, and is not consistently or primarily due to resiliency of the underlying soil or foundation.

Sutherland states "It is probable that a reduction in the magnitude of the corner deflections would result in more nearly elastic action of the subgrade in the vicinity of the

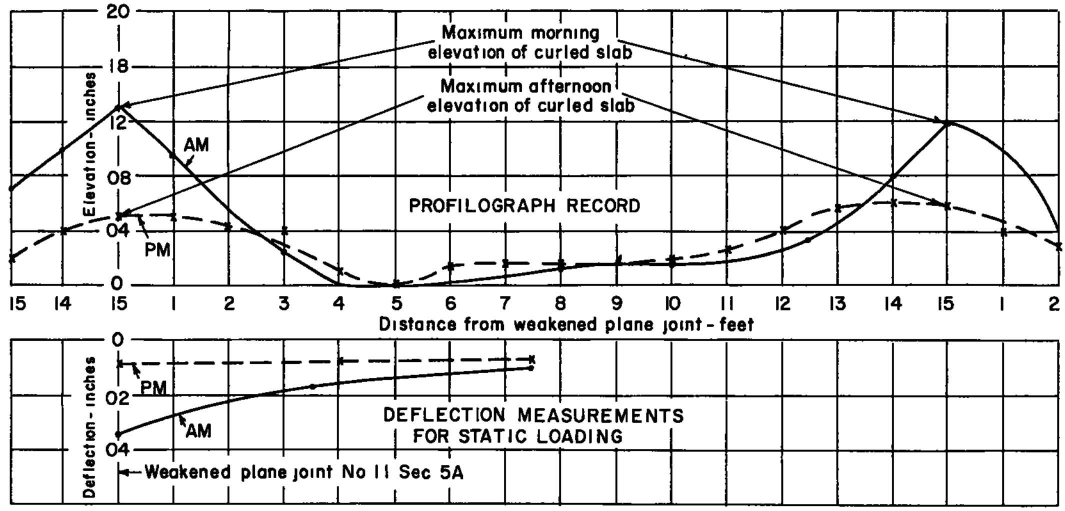

Figure A. Comparison of profilograph record and deflection of slab due to static load.

corner." Figures A and B both indicate that when the curling of the slab is diminished or flattened by the expansion of the upper surface due to direct heat from the sun the slab is then more nearly in contact with the subgrade throughout its entire length and when this condition exists the deflections measured at the end of the slab were found to be little if any greater than at the center.

If it were generally true that the unsupported ends of concrete pavement slabs transmitted significantly greater loads to the subgrade, then the subgrade soil near the ends of the slab should be compressed or compacted to greater density and this should be measurable if any significant compression resulted. Our study of concrete pavements indicated that the reverse is generally the case. In fact, all soils (with the exception of dry cohesionless sands that consolidate from vibration) showed less average density under the slab ends when compared with the average density of the soil under the center of the slab.

A glance at Figures A and B (where it is shown that the slab is in continuous contact with the subgrade only at the mid-point) indicates why this condition should exist. Therefore, in our study of deflections which were primarily aimed at determining the effect of

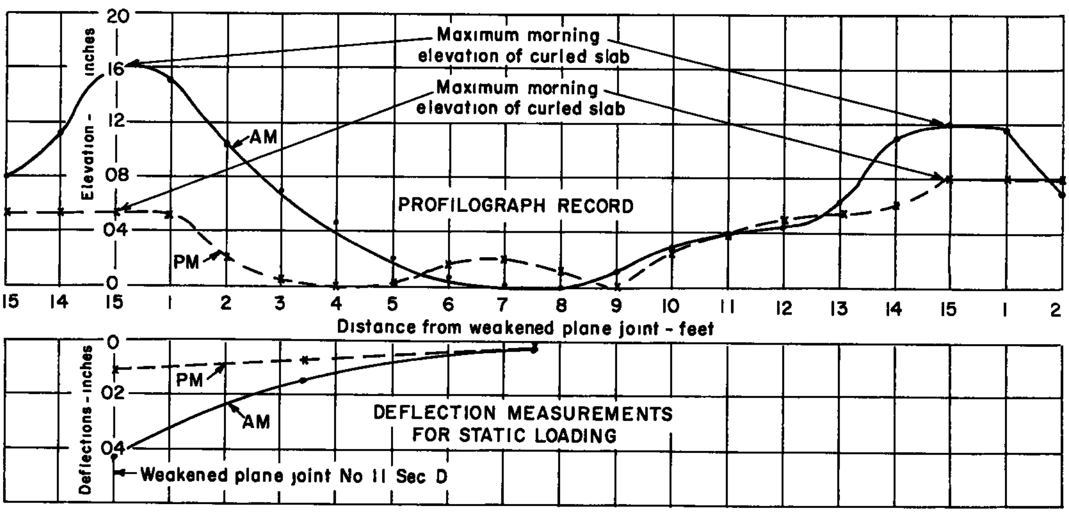

Figure B.

resilience in the soils, we felt that it was necessary to confine the deflection measurements to the midpoint of the concrete slabs.

As illustrated in Table 1, we reached the tentative conclusion that a safe limit for concrete pavement when resting on the subgrade should not greatly exceed 0.012 inch for heavy traffic, and Sutherland indicates that for interior loading the deflections for heavy slabs under loads that would develop transverse bending stresses of half the modulus of rupture were in the order of 0.008 to 0.012 inch. From our viewpoint, this is excellent agreement and it would seem that the data cited by Sutherland tends to support the tentative conclusions suggested by our work. In other words, for an admittedly limited comparison the agreement appears to be surprisingly good.

Sutherland takes the stand that the cracking developed on Road Test One-MD is not a proper measure of load equivalency, because cracking developed only as an aftermath of pumping. He states, however, that the pumping was the result of excessive deflection. He would probably also agree that cracks did not occur with the passage of a single truck but developed with the repetition of loads.

It should be pointed out that pumping action, if we mean the intake and forceful ejection of water from beneath the slabs, could not of itself cause cracking. The pumping of concrete-pavement slabs is significant only to the extent that it removes the subgrade support by washing away the soil, leaving a cavity that permits greater deflection. Therefore, it seems difficult to avoid the conclusion that excessive deflection coupled with load repetition was the primary cause for the cracking on the Maryland road.

Reports on the concrete pavement test by the Corps of Engineers at Columbia, Mississippi, seem to show a similar pattern in which the linear feet of cracks produced in the pavement by the test vehicle shows a relationship both to the magnitude of the load and to the number of load repetitions.

It is our understanding that the Maryland road was selected for test purposes because it was considered to be representative of a large mileage of pavements in the United States. Figure 45 in the subject paper shows that the deflections of a rigid pavement under 32,000-lb. tandem-axle loads were approximately the same as would be produced by a 24,000-lb. single-axle load, and the published data from the Maryland test road indicated that the linear feet of cracking developed there was the same for the 32,000-lb. tandem as for a 24,000-lb. single axle. It was my intention merely to point out this similarity, but I was not ready to propose a method of design that would make the tandem axles less productive of deflection and pavement cracking.

Sutherland points out that deflections caused by loads acting at the interior of a concrete pavement slab bear no direct relation to the magnitude of the stresses caused by the same loads. It is Sutherland's opinion that the amount of cracking should not be used as a criterion of load equivalency and prefers to focus attention upon the stresses rather than upon the amount of movement. No one will question the obvious fact that the breaks or cracks are associated with either the magnitude or repetition of stresses in the slab. Nevertheless, the application of orthodox structural design concepts to pavements has not seemed to be fruitful.

The results of the Arlington experiments were published in 1935-6. Since that time, a great many pavements have been designed and constructed in an effort to avoid the weaknesses shown by that study. But many of these "improved" pavements are deflecting excessively, either pumping or faulting; cracks are developing prematurely. It appears that many engineers are still dissatisfied with the results of much so-called modern pavement design. If this were not true, there would seem to be little justification for the $11-million test road currently proposed by AASHO.

On the subject of flexible pavements, Sutherland states: "The author presents certain data pertaining to the "resilience" of flexible pavements." This statement by Sutherland is not quite correct. In the first chapter of the paper, "resilience" was listed as one of the properties of the basement soil, and at no point did I intend to convey the impression that asphaltic pavements are considered to be resilient. Sutherland states that the Bureau of Public Roads has used the term "elastic action" rather than resilience. No one can question the validity of the term elastic in this connection; but as explained in a footnote in the paper, it is felt that the term "elastic," as commonly applied to structural materials, may be therefore appropriate for virtually all materials such as steel, concrete, glass, etc. However, even engineers are aware that a sponge-rubber mattress feels different to lie on than a concrete slab. Both are elastic, but the concrete is not resilient. In this comparison, asphalt is undoubtedly less elastic than portland-cement concrete. The so-called flexible pavement with a high asphalt content is probably among the least elastic of structural materials.

In any event, there appears to be no essential disagreement between our findings and the data presented by Sutherland showing the compression and rebound characteristics of a base subjected to repeated loadings under bearing plates. These data are interesting; but in order to evaluate the conditions responsible for either good or poor pavement performance, it seems necessary that we reproduce the actual field conditions, the type of contact between the load and pavement, and by use of full-scale loads on pneumatic tires, determine the amount of deflection to which the pavement is subjected by the vehicles which it must sustain many times daily. Also, when load-deflection tests are performed on an existing pavement that has been under traffic for a period of time, it can be reasonably assumed that the pavement and base have already been "conditioned" so that the measurements will represent its current or "normal" behavior.

Sutherland states: "It would appear to be desirable to so design the pavement surface as to enable it to withstand a greater degree of flexing if this could be accomplished without sacrificing stability." We stated that this is one solution. However, there are two other possibilities. Another is to make the pavement and base of high-strength material which will reduce the deflections below a safe limit.

In fact, there are three possible solutions to this problem, as shown in the lower-right corner of Figure 2. One may design the pavement to develop a sufficiently high slab or flexural strength; the thickness of granular materials may be increased so that the weight tends to reduce any flexing or bending due to resilience in the underlying soil, and finally, a very-thin or flexible pavement may give excellent performance over such foundations.

Both theory and experience testify to the possibility of utilizing the solution mentioned by Sutherland. In fact, if the engineer had at his disposal a paving material that was truly flexible and sufficiently tough and durable, some radical changes in pavement design concepts would be possible. We are handicapped by the fact that we have no paving materials that are truly flexible, and the only means of utilizing this quality is to employ very-thin layers comparatively rich in asphalt such as surface treatments, armor coats, etc. Such thin surfaces are easily damaged by heavy traffic and severe weather conditions, but many have given a remarkable performance, testifying that the principle of a thin wearing surface is sound if it can be made sufficiently tough and durable.

I wish to thank Sutherland for his comments and for the interesting data which adds additional support to the observations discussed in the paper.

Campen notes that deflection or elastic deformation has been recognized as a factor in the designing of pavement thickness. He is correct in pointing out the similarity with the concept as expressed by Westergaard in terms of modulus of subgrade reaction. It is, however, not so evident that there has been orderly procedure for utilizing test data or for meeting this problem on any other basis than personal opinion or "judgment".

Campen has expressed surprise that a critical deflection is of such a low order of magnitude. This would seem to indicate that any "allowance" which has been made must have been based upon assumptions, and it is not clear that many design methods have made "allowance" for the effect of load repetition. Also Figure 14 of the paper illustrates that there may be a great difference between static and dynamic deflections.

Campen also comments that these data are "significant to those who used plates for the evaluation of soils for foundation purposes." In order to be applicable data secured by bearing plates or any other method should be based upon soils in the condition in which they will exist for a substantial period after construction. This means that one must virtually construct a portion of the project before knowing what will be entailed in the design; hence, it would be difficult to know whether the project could be financed until it has been constructed, and in the second place, it is also a most difficult matter to prepare even a small area of subgrade and produce in a short period of time the condition of moisture and density which may be typical of the soil after the passage of time.

Campen also states that the resiliometer results which indicate that compressibility increases as the water content is increased above the optimum "is at variance with basic fundamental principles." As stated in the paper we were admittedly somewhat surprised to note this trend. Nevertheless, we have more faith in "basic fundamental principles," rather feeling that it is a contradiction in terms to imply that observed behavior is in conflict with fundamental principles. It seems safer to assume that we need to know more about fundamental principles, especially which principles are applicable, to explain an observed behavior.

Referring to Campen's final comment, we readily agree with the assumption that the compressibility of entrained gases is the most probable source of the resiliency noted. It also seems most probable that the nature or shape of the soil particles has some affect on the resiliency.

Even though this is true it does not appear unreasonable to hope that we could secure an indication or index by testing a relatively small sample of soil in the laboratory. Engineers have long been satisfied to base the design of steel or concrete structures on relatively small samples tested in the laboratory, even though there is ample evidence to show that as the size of the test specimen is diminished the unit tensile or compressive strengths indicated are usually much higher than can be expected in large members, such as beams or columns of the same material. This fact does not prevent engineers from making good use of test data on small specimens.

Maner has presented an interesting account of deflection measurements on the Virginia State Highway Route 7, east of Leesburg. It appears that there is surprisingly close correlation between the magnitude of deflections associated with a satisfactory pavement in Virginia compared with indications in California. Obviously a pavement might show substantial deflection under a 9,000-lb. wheel load when subjected to test, but the pavement as a whole should not be damaged if 9,000-lb. or similar heavy wheel loads were infrequent in the traffic pattern. As indicated by Maner, the high degree of uniformity in the deflections measured seems to indicate that they have executed an excellent design, as the variation and thickness of subbase evidently compensates for variations in the character of the basement soil.

It would be interesting to examine these deflection data further by reference to the length of the depression created by the wheel load. As stated previously, this relationship, suggested by A. C. Benkelman and W. N. Carey, may provide a better index to the destructive effect of deflections caused by wheel loads.

Herner has made some pertinent comments from the viewpoint of someone primarily concerned with airport pavements. Figure 14 of the paper confirms his statement that deflections under standing loads are usually greater than those under moving wheel loads. Figures 9, 10, 11, 12, 15, and 16 confirm his suggestion that the amount of deflection measured by the G. E. Travel Gauge will vary depending upon the depth at which the reference rod is anchored. It is also true, however, that in many cases there is only a small difference between the deflections referenced at 8 feet compared to 18 feet. We also agree that the much lower rate of load repetition on airport pavements means that traffic effects are much less severe than for highway pavements so far as this factor is concerned.

It seems, however, that there is ample justification for being conservative when considering deflections during the design of a pavement. The report just issued on the WASHO test track (this closure was written on November 1, 1955) indicates that failures occurred with deflections not much above those suggested as limiting values by the California study. The WASHO track was less than 2 years old when testing was discontinued, and the net length of time under which traffic was operated was only about nine months. These factors seem to support the idea that present-day pavements subjected to constant repetition of load should not be expected to withstand much deflection if the pavement is to remain in first-class condition for a period of 10 years or more.

I agree completely with Herner's final observation that airport runways do not ordinarily present a serious problem, since load effects from planes are rather minor at high speeds, due both to the speed and to the fact that a considerable percentage of the load is airborne until the plane has reached the taxiway. It is widely known that taxiways are usually the first to show signs of distress, and this serves to point up the fact observed on highways that slow-moving heavy loads confined to the same wheel path are the most destructive of all, and it is under these conditions that deflections will be found to be of greatest magnitude.