LOAD TESTS ON FLEXIBLE SURFACES

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SYNOPSIS

Within the past year or two there has been a greatly increased interest in load tests as a means of measuring the bearing capacity of flexible surfaces Perhaps the most potent factor in this development is the large airport construction program that has played such an important part in national defense The demand for surfaced all-weather runways capable of supporting airplane loads substantially greater than present-day highway loads has made the rational design of such surfaces a particularly pressing problem The cost of the comparatively large yardages of paving involved has necessitated the use of the lower cost surfaces of the flexible type wherever possible in the place of the higher type of rigid pavement

This paper presents the analysis of some of the data obtained from an investigation of load tests on flexible surfaces conducted by the Michigan State Highway Department in 1937 After completion of the field testing program and analysis of the load tests a program of investigation in the laboratory was undertaken for the purpose of correlating bearing capacity as measured under field conditions with physical tests which might be conducted in the laboratory and ultimately serve as a basis of designing such flexible surfaces in advance of their construction These laboratory tests included particularly the stabilometer test and transverse shear test, both of which required a considerable amount of experimentation in developing procedures adaptable to the problem at hand

The field investigation has indicated quite clearly that the problem of measuring the bearing capacity of flexible surfaces is considerably more complex than in the case of building foundations excepting, of course, obviously non-uniform soil deposits It appears that variations in thickness and consolidation of gravel bases and surface courses under ordinary construction methods are considerably larger and certainly more critical than those usually encountered in natural soil deposits. Flexible surfaces are comparatively thin with an abrupt change in physical properties from those of the subgrade involving discontinuities which are not ordinarily encountered All of these conditions have combined to create a situation that has proved to be particularly perplexing and which yields experimental data very difficult to analyze

The following individual factors in the problem are reviewed and discussed in the paper (1) Load Test Results, (2) Bearing Capacity Relations, (3) Soil Resistance Coefficients, (4) Determination of Yield Values by Time-Settlement Relations, (5) Analysis of Load Tests for a Flexible Surface, (6) Analysis of Results by the Yield Value Method, and (7) Correlation of Field and Laboratory Tests

Although load tests appear to be fundamentally sound some special elements are involved which require consideration In the case of pavements greater duplication of test results is required to obtain representative values than in the case of building foundations

Either one of two methods of analyzing load tests may be used with consistent results The bearing-capacity-limit may be determined by soil-resistance coefficients developed in connection with building foundations, or the yield value for each individual test may be evaluated from variation in the rate of settlement for the various load increments

The most important conclusion of immediate practical importance is that rigid bearing areas produce secondary dimensional effects of such a high order of magnitude that they must be eliminated to obtain reliable results in the case of flexible paving surfaces It is, therefore, recommended that immediate consideration be given to the design of the most practical flexible loading device

It is evident from a review of published information and from discussions at the various meetings that our knowledge of the structural behavior of flexible surfaces is inadequate in several important respects There has not yet been accumulated enough experimental data to make it possible to isolate all of the responsible factors and determine their relative importance It is interesting to note, however, that when faced with the immediate and urgent need for a basis of determining the supporting capacity of flexible surfaces the almost unanimous choice of investigators has been to attempt to measure capacity by some form of direct load tests. Tentative research programs have, of course, included supplementary tests and several other ramifications of the problem but load tests appeared to be the most direct method of approach holding promise of more immediate results.

Load tests have been used successfully as a means of determining the bearing capacity of building foundations but nevertheless there has always been considerable controversy concerning the analysis and interpretation of the data. At first thought it might appear that the procedures successfully used in connection with building foundations could be applied to flexible surfaces with little difficulty. One might expect, for example, to encounter less variation in the constructed surfaces and prepared subgrades than would be encountered in natural soil deposits.

Working on this assumption the writer several years ago formulated a rather comprehensive investigation of load tests on flexible surfaces which was subsequently undertaken by the Michigan State Highway Department After completion of the field testing program and analysis of the load tests a program of investigation in the laboratory was undertaken for the purpose of correlating bearing capacity as measured under field conditions with physical tests which might be conducted in the laboratory and ultimately serve as a basis of designing such flexible surfaces in advance of their construction. These laboratory tests included particularly the stabilometer test and transverse shear test, both of which required a considerable amount of experimentation in developing procedures adaptable to the problem at hand.

This program has been carried on during the past four years and some value has been derived from it but nevertheless it has not been successfully concluded Many unanticipated experimental difficulties have been encountered and results have in general been inconclusive The writer has felt, however, that the methods of attacking the problem have been sound and rather than being discouraged is mainly concerned at this time that others who are attempting to use similar methods will encounter similar difficulties and conclude that load tests and their correlation with laboratory tests is a hopeless task.

Feeling that the ultimate solution of the problem can only be accomplished by actually measuring capacity under field conditions and then correlating these data with appropriate laboratory tests, it becomes particularly important to forestall the real danger that exists of load tests falling into disrepute even though they are fundamentally sound

The investigation conducted by the Michigan State Highway Department has indicated quite clearly that the problem of measuring the bearing capacity of flexible surfaces is considerably more complex than in the case of building foundations excepting, of course, obviously non-uniform soil deposits. It appears that variations in thickness and consolidation of gravel bases and surface courses under ordinary construction methods are considerably larger and certainly more critical than those usually encountered in natural soil deposits. Flexible surfaces are comparatively thin with an abrupt change in physical properties from those of the subgrade involving discontinuities which are not ordinarily encountered. All of these conditions have combined to create a situation that has proved to be particularly perplexing and which yields experimental data very difficult to analyze

The presentation of some of these data and their analysis is no more than a progress report and is done at this time in the hope of promoting a better understanding of the physical relationships involved. If successful results are to be obtained there is necessity for more comprehensive tests and greater care and accuracy in observation and analysis than many may have anticipated. It also appears that some revision of previous methods of procedure may be required to measure properly some variable factors that control the structural behavior of flexible surfaces.

LOAD TEST RESULTS

The experiments conducted by the Michigan State Highway Department in 1937 included tests on a gravel base and bituminous surface on both plastic clay subgrades and sand subgrades. They were made on circular bearing areas of three different sizes varying from 50 to 150 sq. in., all tests being carried to complete destruction of the surfaces. The investigation was carried out during the spring "breakup" period when the subgrades would be in their weakest condition. Shearing resistance tests of the clay subgrade were made to determine its bearing capacity independently of the pavement surface and tests were also made to establish the bearing capacity of the sand subgrades

The various test series are identified as shown in Table 1 with independent tests being conducted to measure the capacity of each element of the surface including the complete pavement structure, the gravel base and the subgrade Three locations were tested with different subgrade conditions including a clay fill, a clay cut, and a sand cut. A total of 33 load tests were made with considerable duplication of results and a large number of supplementary tests to determine gradation, density, moisture content and other characteristics of all materials involved were made

TABLE 1
IDENTIFICATION OF TEST SERIES

Series No	Location No	Type of test	Subgrade material	Test numbers included
1	1	Pavement surface	Clay fill	1, 2, 3, 13, 14, 15
2	2	Pavement surface	Clay cut	4, 5, 6, 7, 11
3	2	Gravel base	Clay cut	8, 9, 10, 12
4	3	Pavement surface	Sand cut	16, 17, 18, 22
5	3	Gravel base	Sand cut	19, 20, 21, 33
6	3	Subgrade unconfined	Sand cut	23, 24, 25, 26, 27, 31
7	3	Subgrade confined	Sand cut	28, 29, 30, 32

Procedures used in making the tests and analyzing the results follow those developed in connection with building foundations and have been described in considerable detail in other publications. The loads were applied by means of a hydraulic jack reacting against a heavy duty platform trailer loaded with pig iron Load increments of approximately 20 lb. per sq. in. were applied every half hour A continuous record of settlement was obtained by measurements made immediately before and after each load increment and at intervening time periods.

For the purpose of this report the results of only one typical test series will be discussed, that being Series No. 1 made on the pavement surface over the clay fill subgrade Data for Series No. 1 are shown in Table 2 and Figure 1.

There was considerable variation in thickness of the various elements of the pavement surface In this connection it should be kept in mind that this road was not built as an idealized test section but included all of the variation that would be found in normal grading operations and road mix construction The bituminous surface varied in thickness from 2 7 to 4 0 in , the gravel base from 4 5 to 6.0, and the total thickness from 7.5 to 9.0 in.

The density of the compacted paving materials was determined from undisturbed samples taken from the completed roadway. The bituminous mixture was a dense-graded oil aggregate using a slow-curing asphaltic oil and its density in place varied from 140.5 to 147 1 with an average of 142.7 lb. per cu. ft. The gravel used in the base conformed to standard highway specifications carrying approximately 9 per cent of clay and silt for binder and its compacted density including 3 9 per cent moisture varied from 142 7 to 151 7 with an average of 147.0 lb per cu. ft.

Samples of the clay subgrade were taken over the entire area covered by the tests The shearing resistance varied from 3.5 to 5.5 with an average of 4.5 lb. per sq. in. On the basis that the developed pressure is equal to four times the shearing resistance this clay subgrade should sustain a pressure of 18 lb per sq in without progressive displacement.

The corrected load-settlement diagrams for Series No. 1 are shown in Figure 1 in which the total applied pressure in pounds per square inch is plotted for settlements up to 3 in. The pavement thicknesses at each test location are shown on the right-hand margin Duplicate tests were made on the 50-sq. in plate and triplicate tests on the 150-sq. in. plate. The original tests are shown by dashed curves, while the average results for each size of plate are shown by heavy full lines.

The variation in bearing capacity with the size of bearing area is in general agreement with previous load test results in that for any specific settlement the smaller plates carry the greater pressure. This is clearly shown by the average curves but the variation in results of duplicate tests on one size of area gives evidence of irregularities that may be very difficult to evaluate. This wide variation does not mean, however, that the results are erroneous as it may very logically be due to the variation in thickness, density and stability of the surface mixtures, subgrade bearing capacity, or any combination of these factors

To obtain consistent results with such variations it becomes obvious that any analysis based on the dimensional proper-

TABLE 2

DATA FOR TEST SERIES NO 1

		Test No	Area sq in	Thickness			P/A	Bearing capacity	Settlement Δ
				Oil agg	Gravel	Total			
Location No 1 Clay fill subgrade	Series No 1 Pavement surface	1	50	4 0	4 5	8 5	0 501	160–196	0 77
		14	50	2 7	6 0	8 7	0 501	170–206	0 49
		2	100	3 0	5 0	8 0	0 354	108–164	0.42
		3	150	3 0	4 5	7 5	0 289	80	0 75
		13	150	2 7	5 0	7 7	0 289	88–128	0 44
		15	150	3 0	6 0	9 0	0 289	96	0 64

ties of the loaded area must include sufficient duplication of results to obtain average load-settlement relations for each size of area It is doubtful whether there are enough tests in the data shown in Figure 1 to meet this requirement On the other hand, the time and expense involved in such tests may make a sufficiently large number of tests prohibitive as routine made and the results appeared to be capable of interpretation with certain limitations that will be pointed out

BEARING CAPACITY RELATIONS

It is first necessary, however, to review briefly the method of analysis that has been developed and used successfully in connection with building foundations

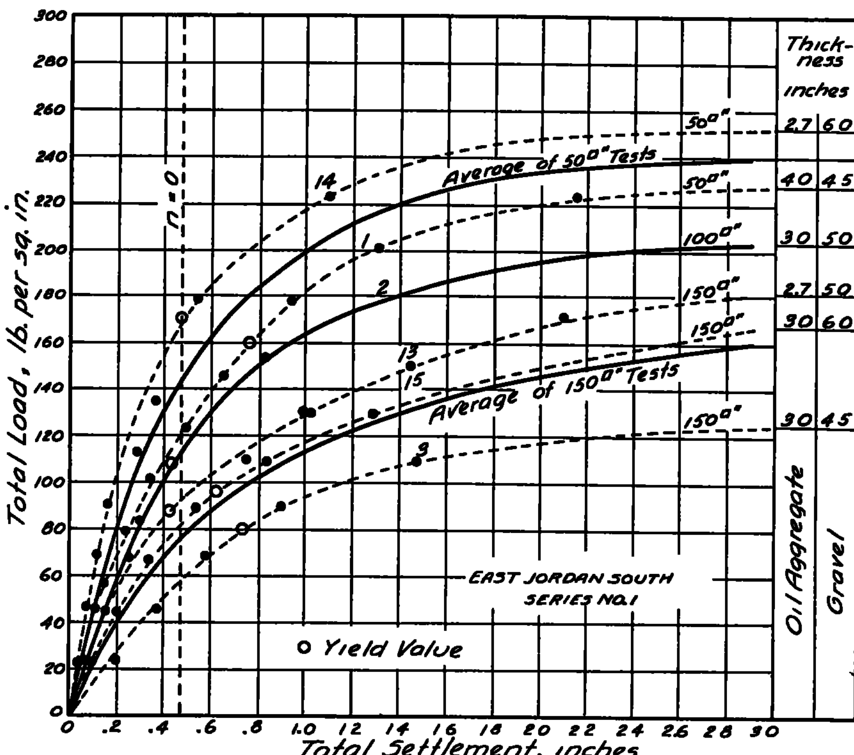

Figure 1. Load Test Results

testing procedure This is a situation that must be faced as an obstacle to be met as part of the problem. It is not, however, a fundamental weakness in load test procedure but is due rather to the inherent irregularities in the materials involved. If the problem is to be mastered enough data must be obtained, otherwise the attempt should be abandoned. Realizing that the data may be deficient, analysis of the load-settlement curves in Figure 1 has been The problem involves the three variable factors of load, settlement, and size of bearing area.

In order to determine the variation in bearing capacity with the size of the loaded area, the loads carried at different amounts of settlement are read from the corrected load-settlement diagrams. An equation expressing this variation has been determined by statistical methods The analyses of some 15 series of bearing capacity tests has indicated that a linear relation between load and perimeter-area ratio, is most satisfactory in reproducing such experimental observations

Such a linear equation has been formulated on the basis that the total load carried by any given bearing area may be expressed as the combined effect of two stress reactions which have been designated as perimeter shear and developed pressure.

This equation is as follows:

$$W = mP + nA$$

in which W = total load in pounds
m = perimeter shear in pounds per lineal foot of perimeter
n = developed pressure in pounds per square foot
P = perimeter in feet
A = area in square feet

Figure 2 illustrates the conditions under which these stress reactions develop support for the loaded area The zone which enters into the support of the bearing area has been commonly referred to as the compression cone. This region has been further divided by designating that portion immediately under the bearing area as the central column. As the bearing area is loaded some of the load is distributed by shearing resistance on the perimeter surfaces to that portion of the compression cone surrounding the central column. It is the accumulation of the shearing resistance acting on the boundary surface which gives rise to the boundary stress reaction which has been designated as perimeter shear Up to the present time the distribution of this shearing resistance with the depth has not been determined. Consequently, this source of resistance is included in the equation for bearing capacity as a concentrated force acting along the boundary and expressed in units of pounds per lineal foot.

As the loading progresses the shearing resistance on the perimeter surface becomes inadequate to continue the lateral transmission of vertical force and additional increments are carried by the central column as developed pressure. This concentration of force in the central column may be continued until the ability of the central column to carry vertical load is exceeded Its supporting capacity arises from two sources. In the first place, vertical concentrations of load may be increased and transmitted downward until the difference between the vertical and lateral pressure exceeds shearing resistance of the soil on inclined planes of maximum shear. In the second place, additional increments may be added without causing failure as long as the lateral pressure furnished by the material surrounding the central column is not exceeded Summation of these several factors are combined in a single stress reaction which has been defined as developed pressure and which is independent of the size of the bearing area.

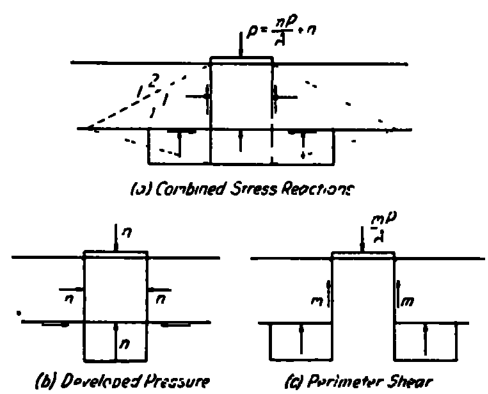

Figure 2. Stress Reactions in the Linear Equation for Bearing Capacity

Inasmuch as it is customary in building practice to deal with bearing capacity in pounds per square foot, the equation for total load may be conveniently expressed in terms of an average intensity of pressure which is defined as the bearing capacity. This equation for bearing capacity may be obtained by dividing both sides of the equation for total load by the area of load application and is as follows:

$$p = m\frac{P}{A} + n$$

In the linear equation which is used in load test analysis the stress reactions m and n are unknowns which are to be determined by the solution of a set of equations representing the load in pounds per square foot carried by the several different bearing areas at any given amount of settlement. Any two equations are sufficient for one determination of the stress reactions m and n It has been found desirable, however, to test three or more bearing areas in order to obtain a better average of these factors. The variation in the stress reactions for the entire range of the tests is determined by the solution of such sets of equations for various amounts of settlement.

In Figure 3 is shown a typical set of load-settlement diagrams for a natural soil deposit including tests on three sizes of bearing area of 1, 2, and 4 sq ft. The accuracy with which the linear equation reproduces the test data may be determined by substitution of the most probable values of m and n and comparing the results with the actual measured load per unit area for each bearing area Such a comparison is shown in Figure 3 wherein the computed values of bearing capacity are plotted against the measured load With the exception of several points in the lower range of load the agreement between the equation and observation is well within an error of 10 per cent, which in addition to checking the validity of the equation indicates the experimental accuracy in test procedure. In Figure 4 are shown the values of the stress reactions m and n for the test series which include the three test areas shown in Figure 3.

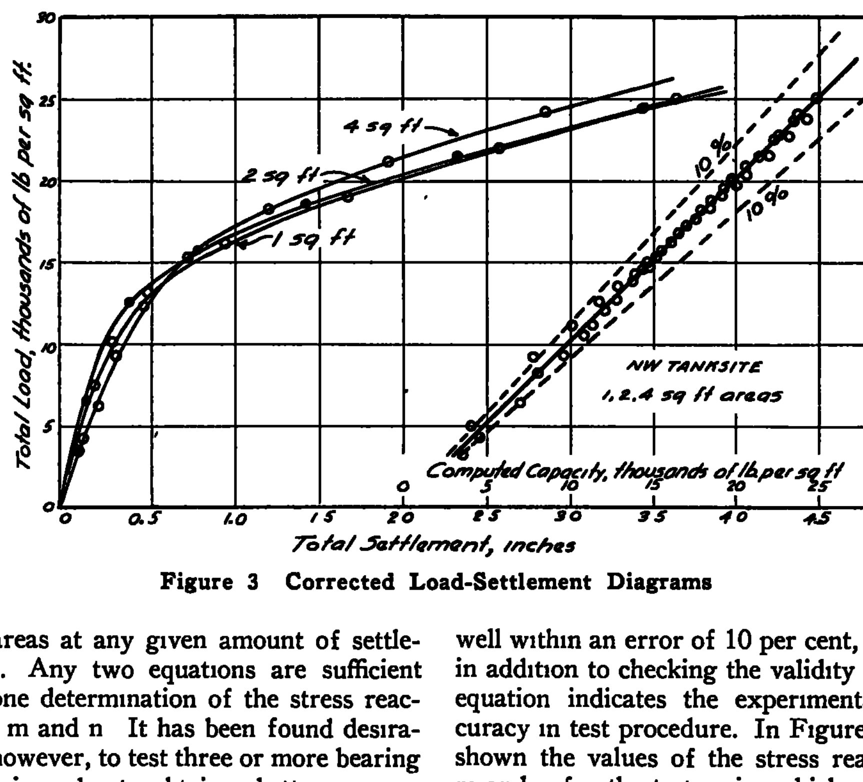

Figure 3 Corrected Load-Settlement Diagrams

SOIL RESISTANCE COEFFICIENTS

The purpose of any analysis of soil test data is to determine the maximum load which will be sustained without excessive settlement. It is enough to consider that there are two stages of soil behavior portrayed by the test data. The first stage is one in which the soil will sustain the applied load without continued or progressive settlement. The second stage is one in which the soil is stressed beyond its yield value, resulting in progressive settlement which continues at an essentially uniform rate as long as the load conditions remain unchanged. The point of transition between these two stages may be defined as the bearing-capacity-limit, the determination of which is the primary objective of the load tests and toward which the analysis is directed. The m and n curves as shown in Figure 4 do not in themselves furnish any criterion for the bearing-capacity-limit of the soil, and it is necessary to turn to the soil resistance coefficients K_1 and K_2 which are derived from the measured values of settlement, Δ, m and n, to define the bearing-capacity-limit. K_1, which is the ratio of settlement divided by developed pressure, is defined as the coefficient of settlement $\left(K_1=\frac{\Delta}{n}\right)$ It is analogous to the well-known coefficient of compressibility except that it expresses the total settlement as volume change in the body of soil included within the compression cone, rather than being expressed in deformation per unit volume. K_2 is the ratio of perimeter shear divided by developed pressure and is defined as the stress reaction coefficient $\left(K_2=\frac{m}{n}\right)$. It expresses the relative importance of the two stress reactions involved in bearing capacity.

The bearing-capacity-limit of the soil may be determined as the minimum value of K_1 or the maximum value of K_2 depending upon the sequence in which the two types of resistance are developed For a relatively compressible soil the developed pressure is small for the lower range of loads and the major portion of the applied load is carried by perimeter shear. As the settlement increases and the bearing plate penetrates the surface, developed pressure increases and the values of K_1 decrease as shown in Figure 4. The decreasing values of K_1 show that the resisting pressure is increasing faster than the settlement and indicate a margin of resistance which is available to bring the loaded area to equilibrium if no more load were to be added. The minimum value of K_1 defines the maximum developed pressure in the case of soils which are relatively compressible. Subsequent increasing values of K_1 in which the settlement increases more rapidly than the developed pressure show that increments of settle-

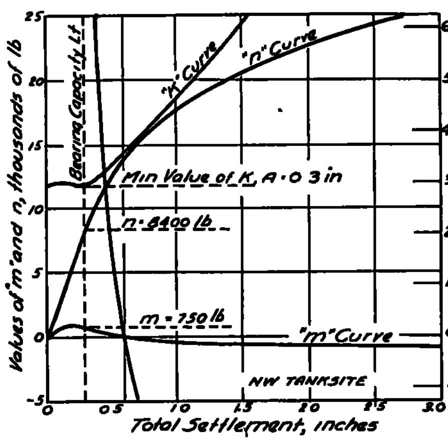

Figure 4. Stress Reactions and Soil Resistance Coefficients

ment are accumulating without proportional increase in resistance and signify the stage of progressive settlement Meanwhile the values of K_2 are decreasing and show no evidence of critical changes in behavior. The supporting capacity due to perimeter shear was available in the initial stage of loading and after having been fully utilized exerts no further influence on the transition to the stage of progressive settlement.

In Figure 4 the bearing-capacity-limit at a minimum value of K_1 occurs at a settlement of 0 3 in. with a value of perimeter shear m equal to 750 lb. per lin. ft and developed pressure n equal to 8,400 lb. per sq. ft. As an illustration of the use of the data in the linear equation the bearing-capacity-limit of the 4 sq ft round plate may be computed as in the following example:

$$p = m\frac{P}{A} + n$$

$$\frac{P}{A} = 1\ 77 \qquad m = 750 \qquad n = 8{,}400$$

$$p = 750 \times 1.77 + 8{,}400 = 9{,}730 \text{ lb per sq. ft}$$

In a similar manner the bearing-capacity-limit or yield value of the 1 and 2 sq ft. plates may be computed.

1 sq. ft Area $\qquad \frac{P}{A} = 3.55$

$$p = 750 \times 3.55 + 8{,}400$$
$$= 11{,}060 \text{ lb. per sq ft}$$

2 sq ft. Area $\qquad \frac{P}{A} = 2.51$

$$p = 750 \times 2\ 51 + 8{,}400$$
$$= 10{,}280 \text{ lb. per sq ft.}$$

DETERMINATION OF YIELD VALUES BY TIME-SETTLEMENT RELATIONS

The bearing-capacity-limit of the soil for any given size of bearing area represents the load at which the yield value of the soil has been exceeded In many of the series of load tests conducted by the writer, independent shearing resistance tests have been made and these results correlated with the results obtained from the analysis of the load tests by use of the linear equation for bearing capacity Such correlations suggest another method of determining the yield value load for any given size of bearing area.

This method is illustrated in Figure 5 and involves determination of the terminal rate of settlement for each load increment applied to the bearing area There is shown in Figure 5 typical time-settlement curves for the 4 sq ft. bearing area in the series of tests in the preceding discussion. The slope of these curves represents the rate of settlement at any time during the 60-min loading period, and the final value at the end of that period is the terminal rate of settlement

It may be noted that the total settlements for the first three load increments of 3,000, 6,000, and 9,000 lb. per sq. ft are practically equal and the final rates of settlement are zero, both of which observations are indicative of elastic behavior. For the increment of 12,000 lb. per sq. ft there is definite indication of progressive settlement and for each succeeding load increment the final slope of the curves or the terminal rate of settlement increases as the load increases. It appears that the

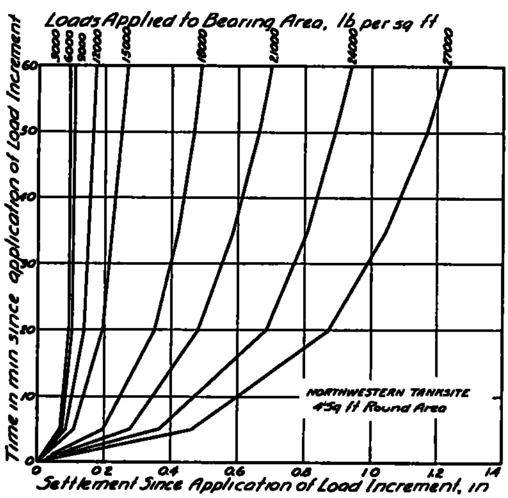

Figure 5. Time-Settlement Curves

bearing-capacity-limit or yield value load would lie between 9,000 and 12,000 lb. per sq. ft and this is verified by the foregoing analysis in which the computed bearing-capacity-limit was 9,730 lb. per sq. ft

It thus appears that the rates of settlement may be used to determine the bearing-capacity-limit or yield value load for each size of area independently The manner in which this may be done is shown in Figure 6 in which the terminal rate of settlement is plotted against the applied pressure in kips per square foot. In each case the rate of settlement is zero for the first two or three load increments after which there is a marked change in behavior with the rates of settlement increasing as the load increases This variation is approximated by a straight line and the intercept on the vertical axis determines the yield value loads which are noted on the figure

The figures in parenthesis are the yield loads computed from the linear equation of the previous analysis and it may be noted that the comparison between the

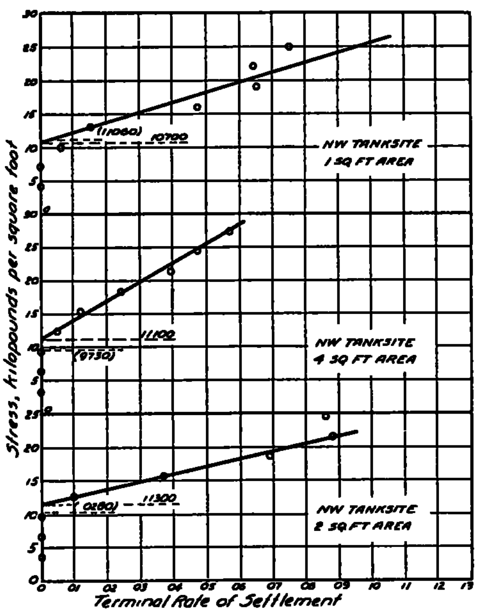

Figure 6. Yield Value Curves

two methods of analysis is reasonably good It is felt, however, that the use of the linear equation gives a more consistent representation of the average capacity of the soil deposit as it includes a statistical average of all sizes of area tested. A similar result could be obtained by solution of a set of linear equations embodying the yield value loads determined independently from rates of settlement for each area tested.

This latter procedure has the disadvantages of being more dependent upon the personal equation in selecting the best straight line variation in the stage of progressive settlement and is more directly affected by any inaccuracy in settlement measurements On the other hand, the determination of independent yield values may be applied to load tests in which the variation in such factors as pavement thickness make it impossible to group all tests into one series for solution of a set of linear equations as has usually been done For this reason alone it is more adaptable to the analysis of load tests on flexible surfaces.

ANALYSIS OF LOAD TESTS FOR A FLEXIBLE SURFACE

It is now proposed to analyze the load tests shown in Figure 1 and designated as Series No. 1 by both available methods for comparison. The results of the first type of analysis by solution of sets of linear equations for equal settlement are shown in Figure 7.

Attention should first be directed to the variation in the perimeter shear, m, and the developed pressure, n, for the entire range of settlement. Up to a settlement of approximately 0 5 in the values of perimeter shear are exceptionally high showing that the load is being carried by shear at the boundary of the bearing areas During this range of settlement the values of developed pressure, n, are negative and changing to positive at slightly less than 0 5 in as indicated by the dashed vertical line where $n=0$ In the higher range of settlement the perimeter shear builds up to a maximum and recedes somewhat near the end of the test. The developed pressure, n, increases throughout the entire second stage of loading in an almost linear variation.

The soil resistance coefficients K_1 and K_2 ordinarily determine the bearing-capacity-limit either at a minimum value of K_1 or a maximum value of K_2. There are no maximum or minimum values in this case except at the vertical asymptote for n=0. The soil resistance coefficients are both negative for settlements less than approximately 0.5 in. and are both positive for settlements greater than that amount. To accept that the maximum or minimum values at n=0 designates the true bearing-capacity-limit would be to place too much faith in a purely mathematical result unless a logical interpretation can be made of the physical relationships involved

On the other hand, it is not difficult to conceive of conditions in which the relative rigidity of the surface or bearing plate would be so great with respect to a soft subgrade that the sequence with which the two dissimilar stress reactions are developed would be reversed. Before inquiring further into this problem it should be pointed out that the results themselves are not fallacious as they represent nothing more than a cross sec-

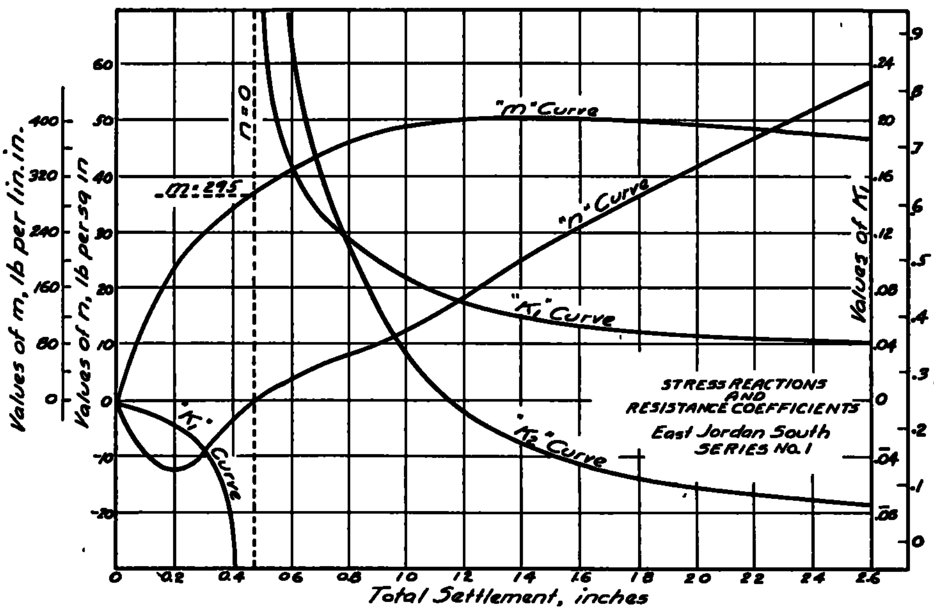

Figure 7. Analysis by Solution of Linear Equations at Equal Settlements

The most critical feature of these unique results is the negative value of developed pressure, n, and the fact that n=0 at the only apparent bearing-capacity-limit. Physically such a result might be interpreted to mean that there was no direct transfer of pressure through the central column of the compression cone to the supporting subgrade, but all load would have to be transmitted through the surface as perimeter shear. Such an interpretation is contrary to all generally accepted ideas of the function of subgrade support which holds that the subgrade must eventually carry the load.

tion of experimental data obtained by carefully controlled tests. As has been pointed out there was wide variation in some physical conditions that could not be controlled which may or may not have been averaged out by duplicate tests Nevertheless the data are worthy of additional examination for several reasons. In the first place, the negative values of n shown for Test Series No. 1 were found in approximately the same magnitude in all of six other test series which have not been presented in this discussion. In the second place, such results have also been found in several series of load tests conducted for building foundations and have not vitiated the results nor interfered with their successful application to the design of spread footings for a predetermined settlement. The data are, however, particularly critical in this case because of the extended range of settlement in which n appears to be negative and because of the fact that there are no other apparent maximum or minimum points.

ANALYSIS OF RESULTS BY THE YIELD VALUE METHOD

Before attempting any final interpretation of negative values of developed pressure it is desirable to determine the yield value loads for the individual tests by analysis of settlement rates, which is the second method proposed

The application of this procedure is illustrated in Figure 8 for a typical test on the 50-sq in plate The time-settlement curves for each load increment are shown at the top of the figure and the relation of terminal rate of settlement and load may be determined as in the previous example. In this case the use of the shorter time period of 30 min. between load increments is not as satisfactory as a longer time period. Loads less than the yield value show small settlement rates due probably to volume change as equilibrium had not yet been achieved When rates of settlement are to be so used, the longer time periods of at least one hour between load increments should be used and settlement readings should be made more frequently at intervening time periods to establish the terminal rate of settlement.

Even though more accurate control of the time element would have been helpful the yield-load curve shown at the bottom of Figure 8 indicates two distinct stages of behavior. The elastic range is approximated by the steeper straight line and the second stage of plastic displacement by the flatter straight lines. There is some uncertainty as to where yielding commenced due mostly to failure to use larger time periods or possibly to inherent irregularities common to granular materials. At any rate two possible plastic flow lines are shown and the yield value at the intersection of the two representative lines could vary from approximately 160 to 195 lb per sq in. The selection in case of poorly defined data is a matter of personal judgment and the practice of using

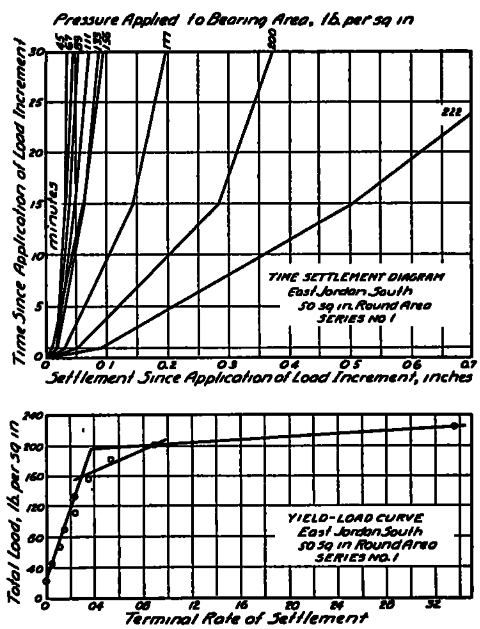

Figure 8. Typical Yield Value Analysis for a Flexible Surface

the lower values is followed as a matter of general practice

The yield loads for all the tests in Series No. 1 have been determined by the method illustrated in Figure 8 and have been tabulated in Table 2 showing also the settlements at which they occur which vary from 0.42 to 0.72 in Referring again to Figure 1 the yield value loads have been shown on the various load-settlement diagrams by circles While there is some variation in the settlements at which these selected yield values occur, they are grouped about the vertical line for n=0 at a settlement of slightly less than 0 5 in. which was suggested as a possible bearing-capacity-limit from the combined analysis using the linear equation for bearing capacity.

Considering the possibility for rather wide variation in behavior of individual tests, this comparison is remarkably good for the two quite different methods of analysis and indicates that both are fundamentally sound. It also indicates that in spite of the experimental difficulties the load tests are apparently a fairly consistent measure of the behavior of the pave-

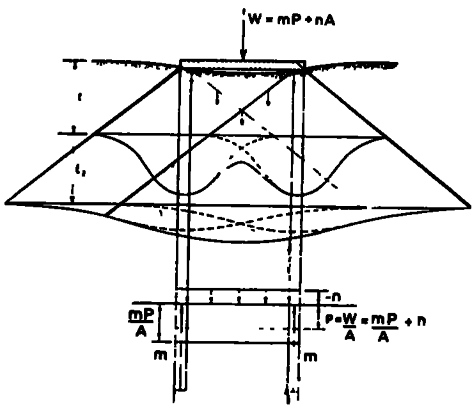

Figure 9. Interpretation of Negative Values of Developed Pressure

ment structure. There remains, however, the puzzling question of negative values of developed pressure Also it is still necessary to break down the composite stress reactions of perimeter shear and developed pressure into their component parts of punching shear and subgrade support in order to effect a correlation with independent laboratory tests which have been proposed.

An interpretation of negative values of developed pressure, n, is illustrated in Figure 9. There is shown a comparatively rigid bearing area which tends to bridge over the elastic depression in the pavement surface created by the total load, W. Bearing is actually developed on a comparatively narrow annular ring at the perimeter of the bearing area Such an edge concentration is obviously a function of perimeter as shown when the variation in bearing capacity for different sizes of area is established by solution of simultaneous linear equations

The distribution of pressure at the pavement surface is shown at the bottom of the figure. When the edge concentration, mP, is expressed as an average pressure by dividing by the area, A, the pressure ordinate obtained is greater than that obtained by dividing the total load, W, by the area In the linear equation this unbalanced condition is compensated by a negative value of n. This result, as previously pointed out, is not fictitious but comes directly from the observed variation in bearing capacity.

As shown by pressure distribution on horizontal planes representing the supporting subgrade for two different thicknesses of pavement, the negative value of n holds only for the pavement surface. Actually subgrade support in the center is required as soon as the pressure cones originating at the perimeter overlap If the ratio of thickness to the diameter of the loaded area is large the maximum pressure transmitted to the subgrade would occur under the center of the area as would normally be expected. The capacity of the surface to carry load would then depend upon subgrade support as the essential element.

It is important to note that the support contributed by the subgrade reaches the loaded area on the surface as a concentrated force at the boundary rather than a pressure transmitted directly upward through the central column. Subgrade support thus becomes involved in a stress variation which rather completely masks its true function. It is significant that this state of affairs has been produced by the use of a relatively rigid plate producing a pressure variation on the loaded surface substantially different than that existing at the contact plane of surface and subgrade

The conclusion can scarcely be avoided that the use of rigid bearing areas has led to the serious complication and the most practicable method of avoiding it is to use flexible loading devices This is not as easy as it sounds as certain difficulties, particularly in measuring settlement are immediately encountered. The proper degree of flexibility is also difficult to determine and control. The ideal condition is a loading device which just matches the flexibility of the surfaces and produces an essentially uniform application of pressure. Pneumatic tires may be the answer if inflation pressures could be adjusted to maintain the same contact area Otherwise they would involve another troublesome variable in an already complex problem.

The writer has come to the conclusion stated in the last paragraph rather reluctantly and until recently was inclined to insist upon rigid bearing plates Justification for this view was found in simplification of settlement measurements and the fact that secondary boundary effects had caused no difficulty in load tests for building foundations. This latter observation can probably be traced to the fact that natural soils are comparatively compressible and uniform bedding of the plates can be obtained without more than negligible stress concentrations at the edge By the same line of reasoning it appears that paving surfaces fall into a different category and these effects should be eliminated.

CORRELATION OF FIELD AND LABORATORY TESTS

The present status of the investigation that has been discussed does not justify more than a few brief notes on the correlation of field and laboratory tests The results obtained to date are not entirely discouraging and a few tentative approximations of the punching shear of the surface and subgrade support may be of interest.

The primary advantage of separate determination of the yield value load for each individual load test is that the variable thickness of the surface may be included in the solution of linear equations for bearing-capacity-limit This is done on the assumption that the perimeter shear, m, is entirely due to punching shear through surface and may be evaluated by the product of the shearing resistance of the mixture times the thickness of the surface The perimeter shear may also be broken down into the two components of the oil aggregate surface and gravel base These relationships may be expressed by substituting the appropriate values of perimeter shear, m, in the linear equation for bearing capacity as follows.

$$p=m\frac{P}{A}+n$$

For combined thickness $m=St$

For separate thicknesses $m=S_1t_1+S_2t_2$

S_1 = shearing resistance of bituminous surface

t_1 = thickness of bituminous surface

S_2 = shearing resistance of gravel base

t_2 = thickness of gravel base

S = average shearing resistance of combined layers

t = thickness of combined layers

The solution of sets of linear equations for the yield value loads of Series No 1 shown in Table 2 and Figure 1 resulted in the following values:

$S=$ 38.7 lb. per sq in.
$n=-2.1$ lb per sq in

$S_1=$ 36.1 lb per sq. in
$S_2=$ 41.1 lb per sq. in.
$n=-3.8$ lb per sq. in.

Using the higher values of yield point load in Table 2 gave the following results:

$S=47.2$ lb per sq. in.
$n=0$ lb. per sq. in.

The value of perimeter shear obtained at the theoretical yield value when $n=0$ and shown in Figure 7 was 295 lb per in. The equivalent punching shear assuming an average total thickness of paving surface of 8.5 in. is 34 8 lb. per sq in.

The ultimate objective of the investigation of bearing capacity of flexible surfaces would be to develop tests which might be performed in the laboratory to serve as a basis for design of such surfaces. Time does not permit a description of this work or presentation of test results which are as a matter of fact still in progress.

Present results have shown that shearing resistance values obtained in the laboratory are of the same order of magnitude as those obtained in the field loading tests. Stabilometer tests have been used to determine lateral pressure components presumed to act on the shearing surface at the perimeter of the loaded area. While these results are fairly consistent the missing link is the determination of probable lateral components acting in the case of the field tests. This problem is complicated by the variable pressure distribution in the central column as illustrated in Figure 9 and until this variation can be evaluated or eliminated by the use of a flexible loading device it is difficult to see how the ultimate objective of a rational design can be achieved.

CONCLUSIONS

As a result of the analysis of load tests on flexible surfaces that has been described the following conclusions are presented at this time

1. Load tests appear to be fundamentally sound as a measure of the supporting capacity of flexible surfaces but do involve some special elements that must be considered.
2. The normal variation in thickness and compactness of paving surfaces as well as in the properties of the prepared subgrade are particularly critical and require a greater duplication of test results to obtain representative values than is the case in building foundations.
3. Either one of two methods of analyzing load tests may be used with consistent results The bearing-capacity-limit may be determined in one case by soil-resistance coefficients developed in connection with building foundations. In the other case the yield value for each individual test may be evaluated from variation in the rate of settlement for the various load increments In the latter method the time element must be carefully controlled and settlement measurements made more frequently The time interval between load increments should be at least one hour and should be constant for all tests
4. The most important conclusion of immediate practical importance is that rigid bearing areas produce secondary dimensional effects of such a high order of magnitude that they must be eliminated to obtain reliable results in the case of flexible paving surfaces It is, therefore, recommended that immediate consideration be given to the design of the most practical flexible loading device
5. Attempts to correlate field and laboratory tests are at least encouraging but several experimental difficulties have yet to be overcome before results obtained in the laboratory can be applied to field behavior.

DISCUSSION ON FLEXIBLE SURFACES

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When a load is applied to a structure, stresses are induced in its various elements and these stresses are accompanied by strains which depend upon the magnitude of the stresses and the stress-strain characteristics of the materials of which the elements of the structure are composed. The accumulative effect of all the strains in the various elements determines the total deformation of the composite structure. In statically indeterminate structures, the development of strain brings about a redistribution of the load-stress relationship (secondary stress effects) which serves to emphasize the interdependency between loads, stresses, and deformations. Any attempt to interpret deformations directly in terms of load without reference to stresses, or through the medium of assumed or hypothetical stress patterns is likely to lead to difficulty. Professor Housel's experience with load tests on flexible pavements as reported in the paper "Load Tests on Flexible Surfaces" is a demonstration of this likelihood

A flexible pavement structure consists of two principal elements, the subgrade and the pavement supported thereon When a load is applied to the pavement the whole structure deforms. The principal resistance to deformation is supplied by the subgrade, because the pavement is relatively thin and has relatively little resistance to cross bending Without doubt the pavement does assist the subgrade in resisting deformation, but it may well be that this part of the total resistance should be ignored in design, just as the tensile stresses in concrete are ignored in reinforced concrete design Future research will be required before a conclusion can be reached on this point. At any rate, since the subgrade is the principal element supplying resistance to deformation, accurate knowledge of the magnitude and pattern of distribution of the stresses on the subgrade is essential to a rational approach to the determination of the supporting strength of a subgrade.

In the writer's opinion, the design of a flexible pavement structure after a design load has been established will consist of three steps. First, the determination of the stress pattern on the surface of the subgrade. Second, the determination of the surface deflection of the subgrade in response to this stress pattern Third, the determination of the safe or allowable deflection of the pavement A lot of research will have to be conducted before adequate procedures for accomplishing these three steps can be recommended, but it is fairly certain that, in any satisfactory flexible pavement design, the essential features of the above three steps will have to be discernible.

Direct load-deformation tests have a wide appeal to those who are interested in the flexible pavement problem The argument seems to run something like this; "since pavement deformation is the thing we are ultimately interested in, why not measure loads and deformations directly and forget about stresses " In my opinion there are only two possible conditions which justify this short-cut type of research One is that in this present national emergency, during which airport runways must be built quickly and on a grand scale, the direct load-deformation tests give results of an immediate and tangible character which are better than nothing at all. Second, if uniform subgrade conditions prevail in a given locality, load-deformation tests in which the test loads adequately represent the actual loads the pavement is to carry, may give results which are reliable in that locality, but which may be inapplicable in other areas where radically different subgrade conditions prevail. However, for a comprehensive flexible pavement design procedure which will be applicable in Maine or California, in Alaska or in Panama, the stress situation as it actually exists in the structure must be taken into account

Professor Housel has predicated his flexible pavement studies on the thesis that the stresses are distributed in the structure in accordance with the "perimeter shear" concept which he has used extensively and with notable success in studying foundation settlements So far as I am aware, the stress situation envisaged by this concept has never been established by direct measurement or observation. Rather it is an hypothetical concept which has yielded remarkably accurate predictions of building foundation settlements and the concept may be amply justified in this particular field on the basis of results obtained by its use But the hypothetical character of the concept must be borne in mind and the validity of its application to another field may need to be reviewed rather critically

In considering the applicability of this concept to the flexible pavement problem, a detailed comparison between foundations and flexible pavement subgrades reveals wide differences in important characteristics which may outweigh the points of similarity between the two subjects Both problems involve the deformation of soil masses under the influence of applied loads, but beyond this statement practically all similarity between them ceases. In the first place foundation loads are applied to the soil over relatively large areas ranging up to several hundred square feet, whereas wheel loads on flexible pavements are applied over areas of the order of magnitude from about ¼ sq ft up to 2 or 3 sq ft Second, foundation pressures are usually applied to strata of soil which are fairly uniform for appreciable depths, whereas a flexible pavement structure consists of a thin layer of very dense soil mixture on top of a natural soil bed which may be relatively yielding in character. Third, foundation loads are applied through rigid footings, while flexible pavement loads are applied, for the most part, through pneumatic tires which are yielding in character and which, in addition to exerting a vertical pressure at the pavement surface, develop certain tangential forces due to the "gripping action" of the tire. Fourth, foundation pressures are applied only once, remaining in place for a very long time, whereas flexible pavement loads are fleeting in character. They are repeated many times during the life of a pavement with varying intervals of time between load applications. Fifth, the foundation soil is usually saturated and usually remains saturated throughout the duration of the load, whereas a flexible pavement subgrade may or may not be saturated and its moisture content will vary more or less widely from year to year and from season to season In addition to the above contrasts, the fact that pavement loads are dynamic in character could be cited However, the problem of static loads on flexible pavements is sufficiently complex to challenge all our attention at present and greatest progress will be made if studies of impact loads are reserved for the future. With these differences in mind we may legitimately question the applicability of the "perimeter shear" concept of stress distribution to the flexible pavement problem even though it may have been proved to be adequate for the foundation problem That it may not be applicable to flexible pavements is strongly indicated by the fact that negative values of "developed" pressure were obtained in Housel's experiments. In his explanation of this unusual phenomenon, Housel indicates in Figure 9 a situation wherein the pressure under the center of the load on a plane at a shallow depth, is less than the pressure below the edges of the load. This imaginary conception is contrary to the facts in every case of measured pressure with which I am familiar

Professor Housel is to be congratulated upon his decision to use a pneumatic load in his future studies of this problem This is a major change in procedure indicated by the third difference enumerated above and in my opinion it is sound I should like to see him go further along this line and apply loads through an actual tire, although it is realized that this type of loading introduces certain difficulties in measuring deformations. But the kind of loads flexible pavements are called upon to carry cannot be duplicated in any other way.

The fourth difference enumerated, namely, that pavement loads are repeated loads of short duration rather than long time loads, indicates the necessity for a study of the stress-strain relationship of subgrade soils on a considerably different basis than that used for foundation soils It is well known that subgrade soils, especially the clay types, are not elastic in the sense that they will recover on the removal of an initially applied load It seems fairly certain, however, that subsequent applications and removal of load will leave progressively less residual deformation. Indeed, the observation of successful flexible pavement structures leads to the belief that the subgrade soil must eventually be transformed into a quasi-elastic material. Otherwise all such pavements would be badly rutted by wheel traffic, unless the traffic were evenly distributed over the width of the pavement, which is very unlikely. We may visualize this by considering an imaginary case. Suppose a subgrade soil has a residual deformation of 0 001 in after the passage of a wheel load. One thousand such loads would produce a permanent deformation of 1 in and such a pavement would soon be badly rutted. This simply does not happen in many cases, and the conclusion is inescapable that the subgrade soil must become practically elastic after the passage of a number of loads.

In studying the supporting strength of subgrade soil, this concept must be considered and the behavior of the soil should be studied in the light of its performance under repeated loads. This writer does not believe that the "yield point" criterion under a single application of load, which has been used by several investigators, is appropriate to the flexible pavement problem.

A basis for the above attitude concerning the behavior of subgrade soils under repeated loads may be cited in the behavior of concrete under repeated loads. It has been shown that when the first load on a concrete specimen is removed, there is a considerable residual deformation in the specimen which becomes progressively smaller as the number of load cycles is repeated. The rate of decrease of this residual deformation is greater for small loads than it is for large loads.

One further point should be mentioned before closing this discussion, and that is that the criterion for safe or allowable deflection should be based upon the ability of the pavement to undergo repeated deformation without material damage rather than upon the so-called yield point of the subgrade soil Observation of flexible highway pavement failures in Iowa indicates that the primary stages of failure are evidenced by excessive permanent deformation along wide shallow ruts on longitudinal elements where traffic is most concentrated Next the bituminous wearing surface begins to show longitudinal cracks distributed across the shallow ruts and the interval between these cracks decreases as more cracks form. Surface moisture may gain access to the stabilized base course through these cracks, which hastens the deterioration of the pavement and transverse cracks appear in the surface, producing "map cracking." Under further traffic, these map sections may loosen and be thrown out by passing wheels and a chuck hole develops, which spreads rapidly in area and deepens quickly as the wheels abrade the exposed base course.

The development of the longitudinal

cracks in the wearing surface is most prevalent in the late winter and early spring presumably when the subgrade soil is weakened by high moisture content A very large share of the cost of maintaining these highway pavements is represented by the surface treatment required to seal the cracks as they appear and manv sections of the surface must be treated several times a year to prevent their extension to the more advanced stages of failure

It seems probable, therefore, that the ability of a flexible pavement to withstand repeated deformations without loss of stability which leads to surface cracking will have to be considered the basis for the limiting deformation of a subgrade, and this may not be related in any way to the yield-point of the subgrade soil