HIGHWAY RESEARCH BOARD

Research Report No. 4B-1948 Supplement

Airport Runway Evaluation In Canada

PART II

1948

PRESENTED AT THE
TWENTY-SEVENTH ANNUAL MEETING

HIGHWAY RESEARCH BOARD

1948

OFFICERS

Chairman	F. V. Reagel
Vice-Chairman	R. A. Moyer
Director	R. W. Crum
Associate Director	Fred Burggraf

EXECUTIVE COMMITTEE

Thomas H. MacDonald	Commissioner, Public Roads Administration
Frederick M. Feiker	Chairman, Division of Engineering and Industrial Research, National Research Council
Hal H. Hale	Executive Secretary, American Association of State Highway Officials
R. H. Baldock	State Highway Engineer, Oregon State Highway Commission, Salem, Oregon
Pyke Johnson	President, Automotive Safety Foundation, 700 Hill Building, Washington, D. C.
Burton W. Marsh	Director, Safety and Traffic Engineering Department, American Automobile Association, Mills Building, Washington, D. C.
Roger L. Morrison	Professor of Highway Engineering and Highway Transport, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan
R. A. Moyer	Research Associate Professor of Highway Engineering, Iowa State College, Ames, Iowa
F. V. Reagel	Engineer of Materials, Missouri State Highway Department, Jefferson City, Missouri
Charles M. Upham	Engineer-Director, American Road Builders' Association, 1319 F Street, N. W., Washington, D.C.
Stanton Walker	Director of Engineering, National Sand and Gravel Association, Munsey Building, Washington, D.C.

EDITORIAL STAFF

R. W. Crum — Fred Burggraf — W. N. Carey, Jr.

PUBLICATION OFFICES

2101 Constitution Avenue — Washington 25, D. C.

HIGHWAY RESEARCH BOARD

Research Report No. 4B-1948 Supplement

AIRPORT RUNWAY EVALUATION IN CANADA PART II

BY

NORMAN W. McLEOD
Engineering Consultant
Department of Transport
Ottawa, Canada

PRESENTED AT THE TWENTY-SEVENTH ANNUAL MEETING

1947

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

WASHINGTON 25, D. C. DECEMBER 1948

DEPARTMENT OF DESIGN

C. N. Conner, *Chairman*
Principal Highway Engineer
Public Roads Administration

TABLE OF CONTENTS

AIRPORT RUNWAY EVALUATION IN CANADA PART II

By

NORMAN W. McLEOD, *Engineering Consultant*
Department of Transport, Ottawa, Canada

SYNOPSIS

Further study of the earlier data provided by the Department of Transport's investigation and of the additional data obtained during 1947, indicates that the ratio of the load carried at one deflection to the load supported at another deflection for a bearing plate of given size, probably varies with the soil type from airport to airport. However, it appears that representative overall average values for these ratios can be employed with reasonable accuracy when required.

Compacting a clay subgrade to 95 percent modified AASHO maximum density, developed a greater supporting value than that of the uncompacted subgrade at 85 percent modified AASHO. The increase in bearing capacity provided by the first 12-inch layer of compacted soil was considerably greater than that given by an additional 12-inch compacted layer.

Subgrade compaction in the field appears to cause the field CBR and cone bearing ratings of the subgrade to increase at a considerably faster rate than the actual supporting value as measured by the plate bearing test. The Housel penetrometer value is affected in this manner to a less serious degree, while the triaxial test seems to more nearly parallel the increase indicated by the plate bearing test.

A definite relationship appears to exist between the deflection, settlement, and elastic deformation, which occurs when subgrades or flexible pavements are subjected to load.

A simple equation is presented for calculating the subgrade modulus at the surface of any given thickness of well compacted granular base course, if the subgrade modulus for the underlying subgrade has been determined.

The value of K in the flexible pavement design equation $T = K \log (P/S)$ has been found to be dependent upon the size of bearing plate employed, when all other variables are constant. This has resulted in some modification of the pavement thickness design charts for airports and highways contained in the paper presented at the 1946 annual meeting.

The angle of pressure distribution through different thicknesses of base course for 30-inch and 12-inch bearing plates is illustrated for the flexible pavement design equation, $T = K \log (P/S)$.

A method for selecting base course materials, and designing flexible wearing course mixtures by means of the triaxial compression test is outlined.

The Department of Transport's investigation of runways at Canadian airports has continued during 1947. Runways at Moncton, New Brunswick, Calgary, Alberta, and the new airport being built at Saskatoon, Saskatchewan, were included in this

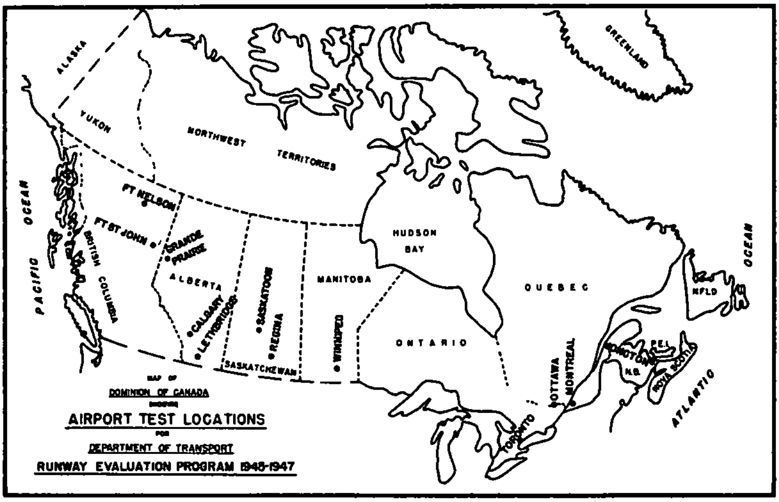

Figure 1.

TABLE I

GENERAL DESCRIPTION OF AIRPORT SITES

Airport	Depth to Water Table	Pavement	Base Course	Sub-Base	Subgrade PRA Classification	Subgrade L.L. Ave.	Subgrade P.I. Ave.
Calgary	Generally Deep	2.5 to 3.0 in. SC-5 bituminous mixture	3.5 to 8.0 in. crusher run gravel	Nil	A-4 A-6	29.1	9.1
Saskatoon (No. 2)	Deep	3.5 in. Asphaltic Concrete	12 in. Mechanical Stabilization	Nil	A-7 A-6	40.3	20.7
Moncton	Deep	3 to 4 in. SC.5 Bituminous Mixture	3 to 4 in. crusher run gravel	8 to 19 in. sandstone 3 to 5 in. gravel	A-6 A-4	29.3	11.7

year's testing program, (Fig. 1). The general description of the subgrade, base course, and pavement for these three airports is contained in Table 1.

1. *Relationships Between Loads Supported at Different Deflections on a Given Bearing Plate*

Figure 2 indicates the ratio of the load carried at any specified deflection from 0 to 0.7 inch for 10 repetitions, to that supported at the deflection of 0.2 inch, for subgrade load tests on a 30-inch diameter plate, for each of eleven airports. The broken line curve represents the overall average values for these ratios for the eleven airports.

While there is an individual set of ratios for the subgrade load tests with the 30-inch plate at each airport, the range of variation at 0.5 inch deflection is from 88 to 108 percent of the overall average value. That is, the average curve provides values for bearing capacity that are within 12 percent of the actually measured values at 0.5 inch deflection for these eleven airports. The maximum deviation from the overall average curve is somewhat greater than this for deflections larger than 0.5 inch.

Similar information for load tests made on the surface of flexible pavements with a 30-inch bearing plate at 11 airports is given in Figure 3. Here the range of variation at 0.5 inch deflection is from 85 to 121 percent of the overall average value.

In general, the curves for each airport in Figures 2 and 3 occupy similar

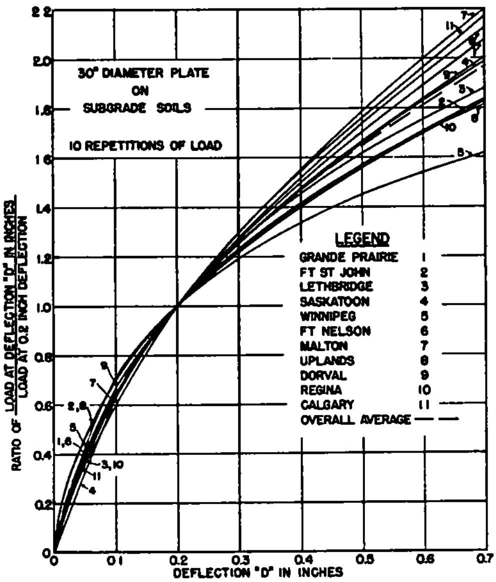

Figure 2. Ratio of load at deflection "D" in inches over load at 0.2-in. deflection versus deflection "D" in inches for 30-in. diameter bearing plate. (Subgrade)

relative positions with respect to the overall average curve in each diagram. For example, curve No. 5 for Winnipeg is the lowest curve in both figures. Also, curves Nos. 2, 3, 5, 8 and 10, for Fort St. John, Lethbridge, Winnipeg, Uplands, and Regina, are below the overall average curve in each figure.

The overall average ratios shown in Figures 2 and 3 were determined by the method of least squares. Overall average ratios were obtained in a similar manner

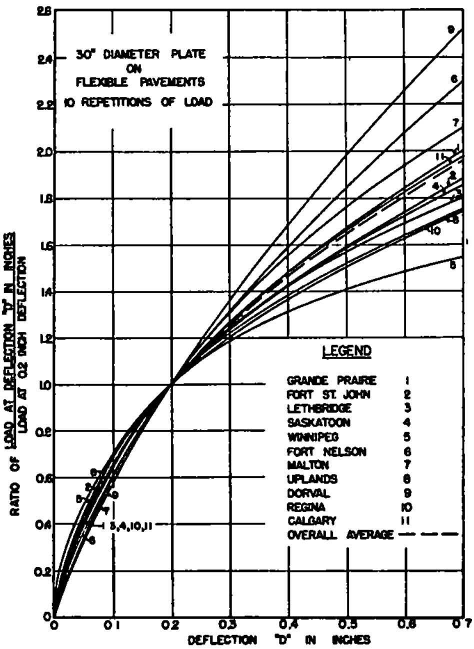

Figure 3. Ratio of load at deflection "D" in inches over load at 0.2-in. deflection versus deflection "D" in inches for 30-in. diameter bearing plate. (Flexible Pavements)

for other sizes of bearing plates. These relationships are summarized in Table 2 for load tests on subgrades, and in Table 3, for load tests on flexible pavements.

Probably the most striking fact illustrated by the data of Tables 2 and 3, is that for any given size of bearing plate, the ratios for any two specified deflections are very nearly identical for both subgrade and surface load tests.

Data for the 18- and 42-inch bearing plates were thought to be too limited to provide representative average information for inclusion in Tables 2 and 3.

It should be emphasized that the ratios of Tables 2 and 3 will probably apply only for the particular load test procedure employed throughout the Department of Transport's investigation.

TABLE 2

SUBGRADE LOAD TESTS

Average relationship between load carried at any deflection to the load supported at 0.2 inch deflection for bearing plates of different diameters.

Diameter of Bearing Plate Inches	Ratio of		Load Supported at Deflection "D" / Load Supported at 0.2 inch Deflection					
	0.05/0.2	0.1/0.2	0.2/0.2	0.3/0.2	0.4/0.2	0.5/0.2	0.6/0.2	0.7/0.2
12	0.372	0.634	1.000	1.253	1.443	1.594	1.724	1.838
24	0.398	0.658	1.000	1.244	1.437	1.601	1.751	1.889
30	0.394	0.651	1.000	1.253	1.462	1.646	1.811	1.966
36	0.392	0.638	1.000	1.263	1.479	1.675	1.850	2.015

TABLE 3

SURFACE LOAD TESTS ON FLEXIBLE PAVEMENTS

Average relationship between load supported at any deflection to the load supported at 0.2 inch deflection for bearing plates of different diameters.

Diameter of Bearing Plate Inches	Ratio of		Load Supported at Deflection "D" / Load Supported at 0.2 inch Deflection					
	0.05/0.2	0.1/0.2	0.2/0.2	0.3/0.2	0.4/0.2	0.5/0.2	0.6/0.2	0.7/0.2
12	0.368	0.637	1.000	1.242	1.425	1.580	1.718	1.839
24	0.364	0.627	1.000	1.268	1.483	1.668	1.837	1.977
30	0.388	0.641	1.000	1.256	1.467	1.650	1.811	1.963
36	0.388	0.641	1.000	1.258	1.479	1.678	1.836	1.977

2. *Yield Point Deflections for Subgrades and Flexible Pavements*

From subgrade load test data for test locations where bearing plates of more than one diameter were used, the relationships between the unit load carried on bearing plates of different sizes at each deflection from 0.05 to 0.7 in., have been determined by the method of least squares. The results are similar to those shown in Figures 34, 35 and 36 (Fig 38 as published), of last year's paper. Similar results have been worked out for all surface load test locations where different sizes of bearing plates were employed.

From this information, combined with that in Tables 2 and 3, the diagram of Figure 4 for subgrades, and the diagram of Figure 5 for load tests on flexible surfaces, have been prepared. Figure 4 is based upon the unit load supported by a 30-in. diameter plate on a cohesive subgrade for 0.2 in. deflection as unity, from which the unit load for a bearing plate of any diameter of at least 12 to 42 in., and for any deflection from 0 to 0.7 in., can be calculated. Figure 5

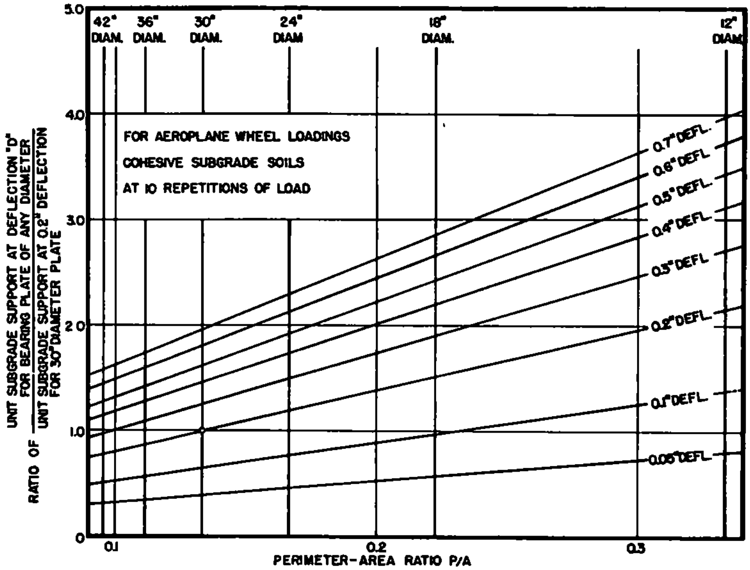

Figure 4. Ratio of subgrade support at deflection "D" for bearing plates of any diameter over subgrade support at 0.2-in. deflection on 30-in. diameter plate versus perimeter area ratio

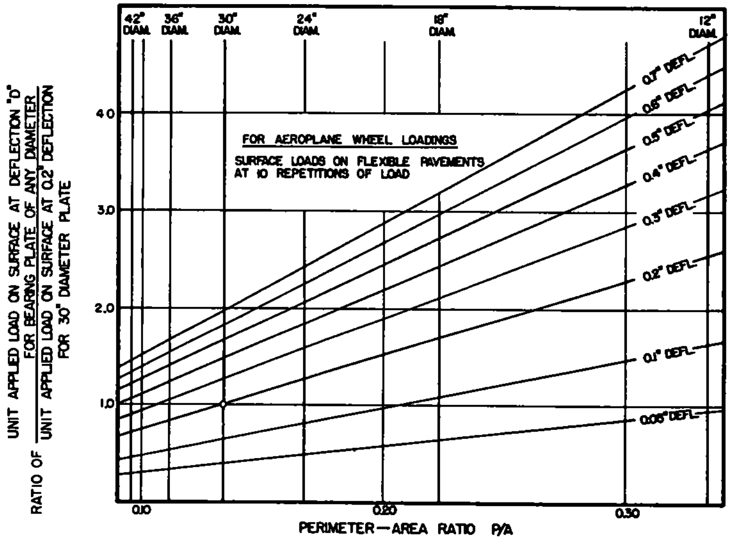

Figure 5. Ratio of applied load on surface at deflection "D" for bearing plates of any diameter over applied load on surface at 0.2-in. deflection on 30-in. diameter plate versus perimeter area ratio

provides similar information for load tests on flexible pavements.

The contact areas of highway wheel loads vary considerably between the smallest passenger car and a large truck, and this difference is even more pronounced

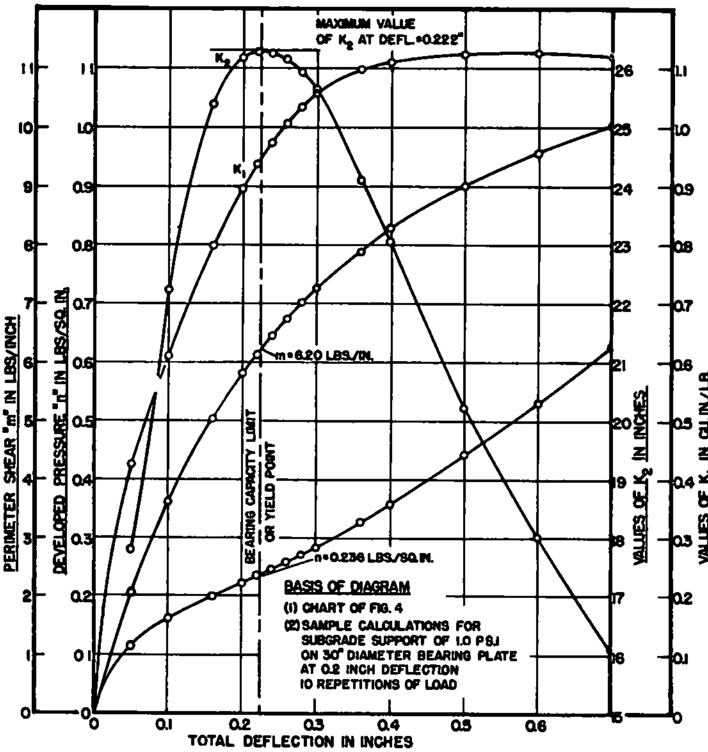

Figure 6. Yield point diagram for cohesive subgrades

between the tires of a small airplane, and those of huge aircraft weighing from 100 to 200 tons that may be relatively common in the near future. It is not practical to have to measure the supporting value of the subgrade for each of these sizes of contact area when working out a design problem involving different wheel loads and tire pressures. Consequently, there is need for a method whereby limited load test data obtained for one size of bearing plate, can be extrapolated to any other contact area for which bearing capacity data are required. Figure 4 answers the need for this information with regard to the subgrade, and Figure 5 provides similar information for load test data on bituminous pavements.

Figures 4 and 5 can be analysed for "yield point" or "bearing capacity limit", by a method devised by Professor Housel (*1,2*)[1]. The results of this metho of analysis are illustrated in Figure for subgrades and in Figure 7 for flex ible pavements. The yield point occur at a minimum value of the K_1 curve, or a a maximum value of the K_2 curve. Figur 6 indicates that the average yield poin for the subgrades at the thirteen air ports tested so far occurs at a deflectio of 0.222 in.

The average yield point deflection fo the flexible surfaces at these airport is shown to be 0.35 in. in Figure 7 According to Professor Housel's inter pretation, the load supported at th yield point deflection is the larges that can be carried without progressiv settlement occuring.

[1] Italicized figures in parentheses refer to the list of references at the en of the paper.

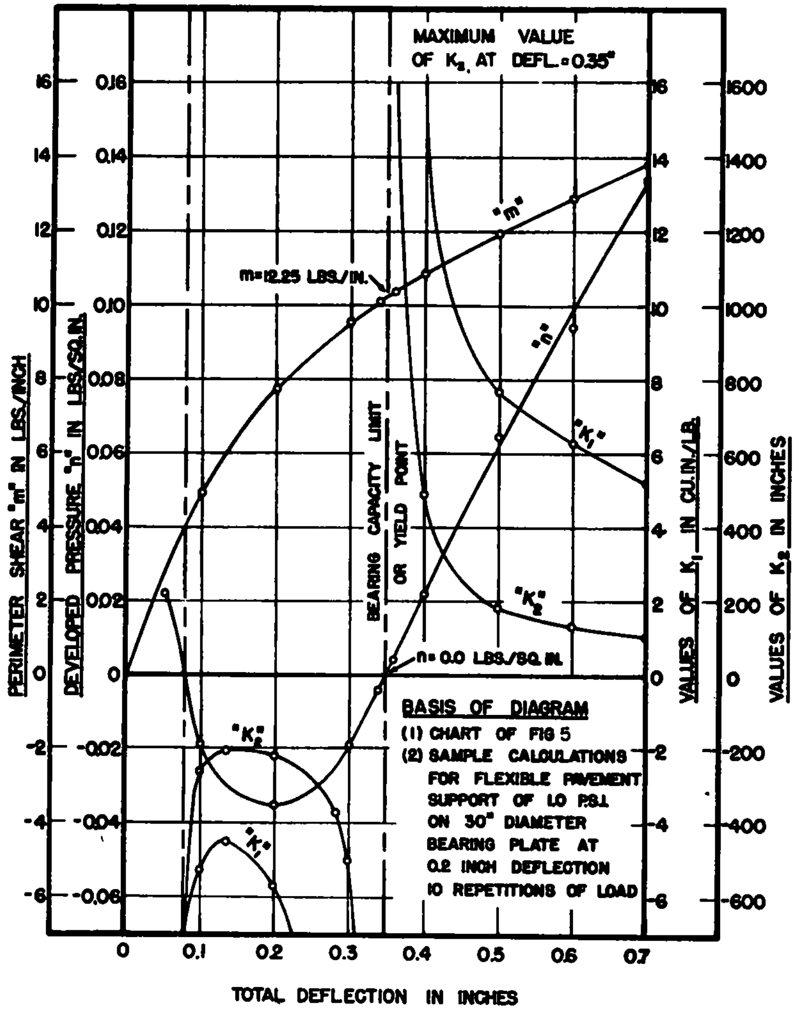

Figure 7. Yield point diagram for flexible pavements

Figures 4, 5, 6 and 7 are based upon all the load test data obtained to date in the Department of Transport's investigation. The results summarized in each of these four figures were determined by analysing all data by the method of least squares.

The yield point deflections indicated in Figures 6 and 7 are in approximate agreement with those given in last year's paper, although last year's values were based on a smaller amount of information, and the method of least squares was not employed throughout when analysing the available data.

3. *Relationship of Unit Load on 12-inch and 30-inch Bearing Plates*

Figure 4 demonstrates that for a range

of deflection from 0.2 to 0.5 in., the unit load supported on a 12-in. plate is somewhat more than twice that supported on a 30-in. bearing plate. For load tests on flexible pavements, Figure 5 shows that for the same range of deflection, the ratio of unit load supported on a 12-in. to that carried on a 30- in. bearing plate is somewhat greater than 2.4.

These two ratios were indicated by data previously reported (*1*). The fact that they are confirmed by the greater number of load tests upon which Figures 4 and 5 are based, is pointed out here, because these two ratios are extremely useful when interpolating or extrapolating limited load test data to contact areas of different sizes. These calculations must always be made when evaluating the runways at an existing airport, or when designing new runways, by means of load test data.

These ratios may not hold for all subgrades and flexible pavements, and wherever possible they should be checked before they are employed for any given project.

4. Relationships Between Deflection, Settlement, and Elastic Deformation

The terms "deflection" and "settlement" are generally used synonymously. In this section however, they will have the different meanings illustrated by Figure 8. When a subgrade or flexible pavement is subjected to repeated applications of a given load, it is depressed below its original elevation when the load is applied. When the load is completely removed it recovers in part, but does not reach its original elevation. The term "deflection" is applied to the amount of vertical deformation which occurs when the load is acting, while the residual vertical deformation remaining after the load is removed is designated "settlement". The difference between deflection and settlement as a load is applied and then removed is defined as "elastic deformation".

If the same load is repeated a number of times, both the deflection and settlement gradually increase. Figure 8 illustrates the gradual increase in deflection and settlement which occurred when a load of 40,000 pounds on a 30-in. diameter plate was applied and released 100 times. The end point deflection rate for both application and release of load was 0.001 in. per minute or less for each of three successive minutes. It appears from Figure 8 that the relationship between number of repetitions of load and both deflection and settlement can be expressed as a straight line on a log-log graph. Figure 8 illustrates the fact that elastic deformation is the difference between the deflection and settlement measurements for any given number of repetitions of load.

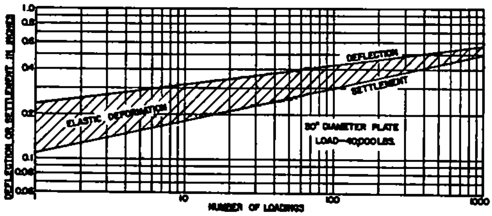

Figure 8. Increase in deflection and settlement resulting from repetitions of a given load

Repetitive load tests have been employed for the Department of Transport's investigation since its beginning. Deflection and settlement measurements for each application and each release of every load respectively, were entered in field note books at the end of every minute as a routine matter. From these data it is possible to establish the relationship between deflection, settlement, and elastic deformation for both subgrade and surface load tests for each airport, and to determine overall average relationships for all airports investigated up to the present time.

In this investigation each load was applied and released from four to six times. The deflection and settlement data for these repeated loadings can be extrapolated to 10 repetitions of load with reasonable accuracy by means of either semi-log or log-log graphs of deflection versus number of repetitions of load, e.g. Figure 8, and probably with less accuracy to 100 or more repetitions.

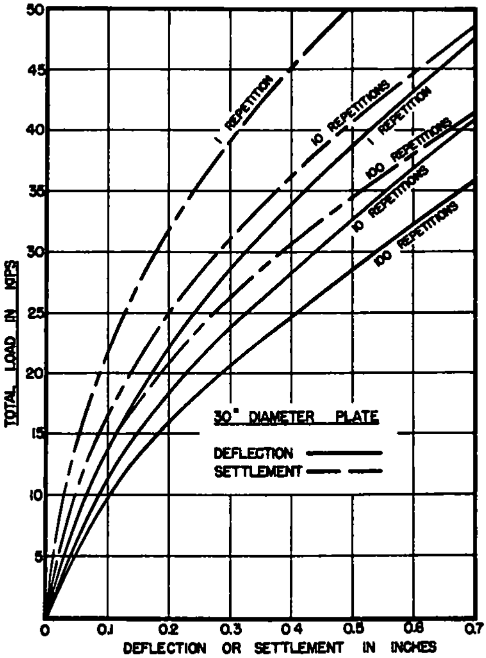

Figure 9. Load versus deflection and settlement for 1, 10, and 100 repetitions of load

Figure 9 illustrates the general nature of the graphs obtained by plotting load versus both settlement and deflection for a single test location. Figure 9 provides information for 1, 10 and 100 repetitions of load.

In Figure 10 the settlement expressed as a fraction of the deflection is plotted versus deflection from subgrade load test data for nine airports having cohesive subgrade soils. The broken line curve represents the overall average relationship for these nine airports. It is apparent that the ratio of settlement to deflection tends to increase as the deflection under load increases. The ratios for the individual airports are all reasonably close to the overall average ratio. At 0.2 in. deflection the maximum deviation is 15 percent, and at 0.5 in. deflection the maximum deviation is 9 percent.

The overall average relationship of Figure 10 has been utilized in the preparation of Figure 11, which indicates the average value of settlement and elastic deformation, for any given deflection from 0 to 0.7 in. for cohesive subgrade soils. Figure 11 shows that for

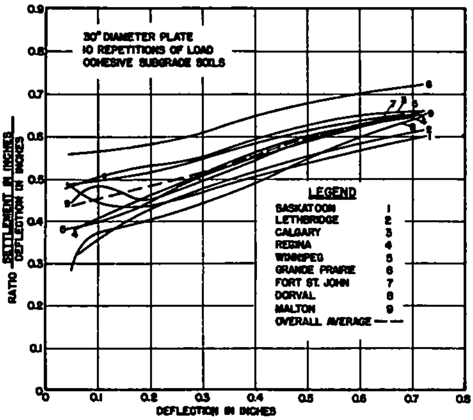

Figure 10. Relationships between deflection and settlement for cohesive subgrade soils

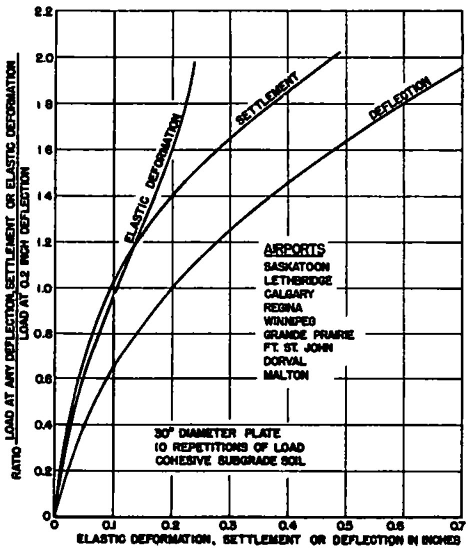

Figure 11. Relationships between deflection, settlement and elastic deformation (cohesive subgrade soils)

a deflection of 0.2 in. caused by 10 repetitions of load on a 30-in. bearing plate on a cohesive soil, the corresponding settlement and elastic deformation will each be approximately 0.1 in. on the average. When the deflection is 0.5 in. under these conditions of load, the corresponding settlement and elastic deformation will average about 0.3 and 0.2 in., respectively. Figure 11 also indicates that to obtain an elastic deformation or a settlement of 0.2 in., the load required is respectively 1.6 times and 1.4 times that which will give a deflection of 0.2 in.

hesive subgrades themselves.

Figures 14 and 15 provide similar information for subgrade load tests at the only two airports with sand subgrades. A comparison of Figure 14 with Figure 10, shows that the ratio of settlement to deflection is considerably higher for these two sand subgrades, then for the subgrades of cohesive soils. For this reason the overall average relationships of Figure 15 for sand subgrades are noticeably different from those of Figure 11 for co-

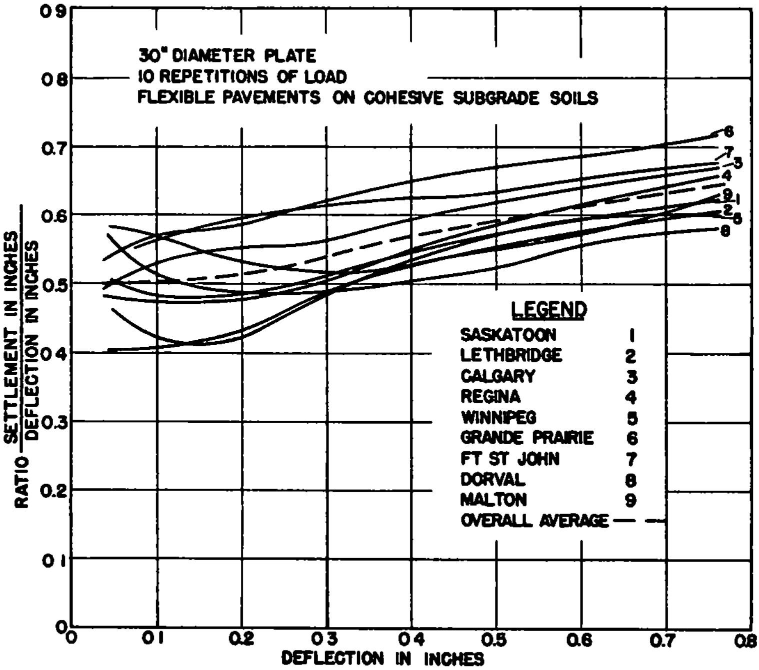

Figure 12. Relationships between deflection and settlement for flexible pavements on cohesive subgrade soils

The relationships between deflection, settlement, and elastic deformation for load tests on flexible surfaces for the nine airports with cohesive subgrade soils are summarized in Figures 12 and 13. The relationships of Figures 12 and 13 approximate those of Figures 10 and 11 for cohesive subgrades very closely, although for the lower deflections, the ratio of settlement to deflection is somewhat smaller for flexible pavements on cohesive subgrades, than for the co-

hesive subgrades. There is a remarkably small deviation from the average ratios for the data in Figure 14, although the two airports with sand subgrades, Uplands at Ottawa in Eastern Canada, and Fort Nelson in Northern British Columbia are about two thousand miles apart.

The relationships between deflection, settlement, and elastic deformation for load tests on flexible pavements over sand subgrades, as summarized in Figures 16 and 17, are in remarkably close agreement with those shown in Figures 14 and 15 for load tests on the sand subgrades themselves.

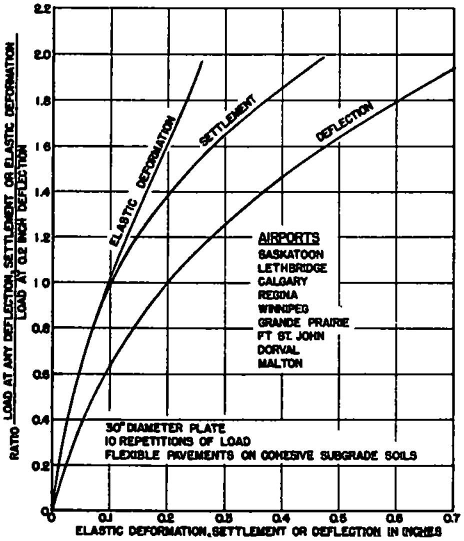

Figure 13. Relationships between deflection, settlement and elastic deformation (flexible pavements on cohesive subgrade soils)

The information outlined in Figures 10 to 17 pertains to 10 repetitions of load on a 30-in. diameter bearing plate. There has not been time to completely

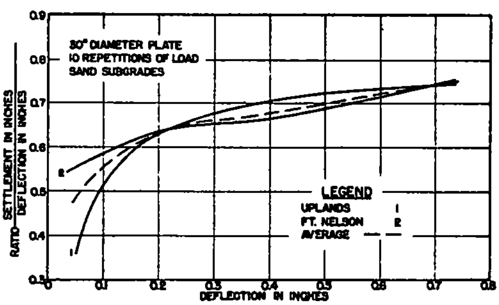

Figure 14. Relationships between deflection and settlement for sand subgrades

analyse these relationships for 1 repetition and for 100 repetitions of load, and for other bearing plate sizes. The limited information so far obtained, however,

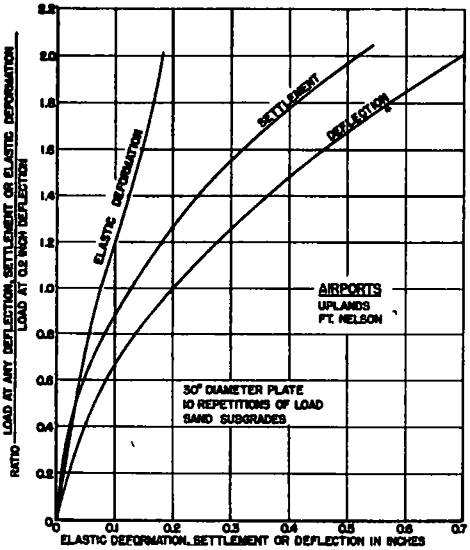

Figure 15. Relationships between deflection, settlement and elastic deformation (sand subgrades)

indicates that ratio of settlement to deflection for 1 repetition of load is about 90 percent of that for 10 repetitions. For 100 repetitions it is about 108 percent of that for 10 repetitions of load.

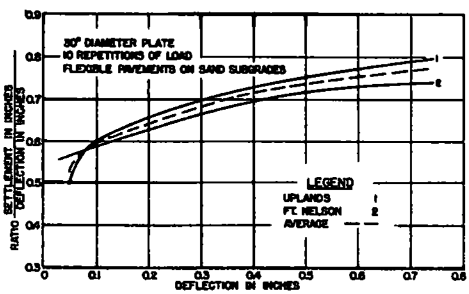

Figure 16. Relationships between deflection and settlement for flexible pavements on sand subgrades

The data analysed so far also tend to indicate that the magnitude of these ratios is greater for a smaller size of bearing plate than for one of larger diameter.

The question might be asked, what is the practical value of the average relationships between deflection, settlement and elastic deformation summarized in

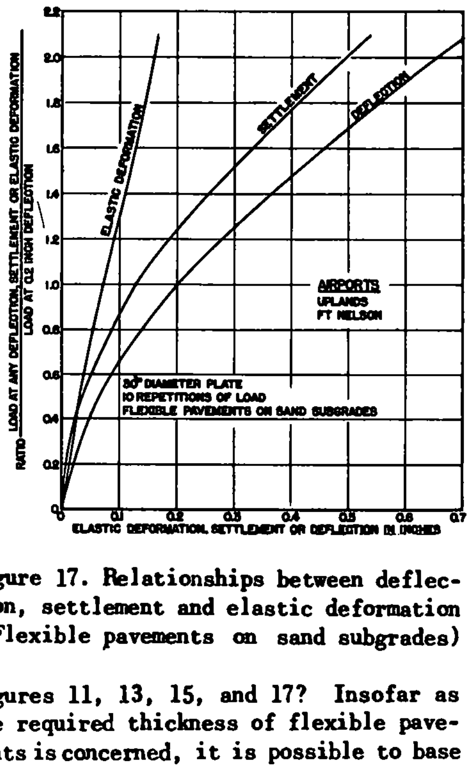

Figure 17. Relationships between deflection, settlement and elastic deformation (Flexible pavements on sand subgrades)

Figures 11, 13, 15, and 17? Insofar as the required thickness of flexible pavements is concerned, it is possible to base their design upon some value of settlement or elastic deformation rather than upon a critical deflection. The average relationships shown in Figures 11, 13, 15 and 17, tend to indicate that little would be gained by so doing. That is, on the basis of the information contained in Figure 13, for instance, it appears to make no difference whether design is based for example on a deflection of 0.5 in., a settlement of 0.3 in., or an elastic deformation of 0.2 in., all at 10 repetitions of load, since the same applied load is required to give each of these three values.

Since deflection is ordinarily an easier property to measure, and a more simple concept to understand, and because for any given applied load, the amounts of settlement and elastic deformation are approximately definite fractions of the deflection, there may be no advantage in considering the design of thickness for flexible pavements in terms of elastic deformation, or settlement, in place of deflection.

It has been indicated by some investigators that after a certain small number of repetitions of any given load, a soil becomes perfectly elastic in its behavior (*3, 4*). If it reaches this point, a constant value of deflection will occur each time the load is applied, and it will recover to a constant value of settlement when the load is released. This would result in a constant value of elastic deformation also. It is to be observed that this condition of elastic behavior does not seem to develop for the testing procedure and magnitudes of load and deflection employed for the Department of Transport's investigation, at least not up to 100 repetitions of load, e.g. Figure 8. While for normal testing procedure only 6 repetitions of each load were employed, individual tests were made at a number of airports in which the same load was applied and released from 80 to 100 times.

5. *Influence of Subgrade Compaction on Subgrade Bearing Capacity*

For a runway constructed in 1946 for a new airport at Saskatoon, Saskatchewan, the top 2 feet of the clay subgrade (L.L. = 35-50%; P.I. = 20-25%; Pass No. 200 = 65-85%) were compacted to 95 percent of modified AASHO maximum density under rigid field laboratory control. A dense graded, impervious, mechanically stabilized base course was constructed on the subgrade, and the asphaltic concrete surface was partly completed. No surface water could penetrate through to the subgrade. After standing during the winter, and being exposed to one complete yearly cycle of weather conditions to permit reasonably uniform distribution of moisture through the subgrade, plate bearing tests were made at seven different locations on the runway during the summer of 1947, to determine the influence of compaction on subgrade bearing capacity.

Pavement and base were removed, the subgrade was excavated as required, and repetitive unconfined load bearing tests were performed with a 24-in. plate on the top of the uncompacted subgrade, on top of 12-in. of compacted subgrade, and on top of 24 in. of compacted subgrade, Figure 18.

Figure 19 indicates that compacting a 12-in. depth of this soil from 85.5 to 94.4 percent of maximum density, increased the subgrade bearing capacity by approximately 25 percent, while compaction to a depth of 24 in. increased the bearing capacity of the subgrade by about 38 percent.

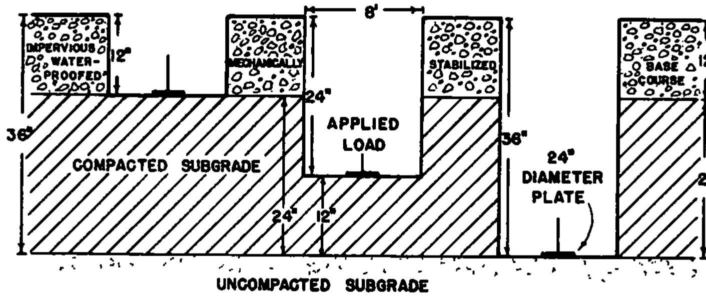

Figure 18. Arrangement of load tests on compacted and uncompacted subgrade

Field moisture and density tests on each 6-in. layer throughout the 24-in. depth of compacted material were made at each load test location in 1947. They indicated that the density in place of the compacted subgrade averaged 94.4 percent of modified AASHO maximum, and the moisture content averaged 114.2 percent of modified AASHO optimum. The corresponding average in place density and moisture contents of the underlying uncompacted subgrade as determined for each 6-in. layer to a depth of 24-in. were 85.5 percent of maximum, and 118.0 percent of optimum respectively.

The load test data have been plotted in Figure 19 for 0.5 in. deflection and 10 repetitions of load. Two principal conclusions can be drawn from the data of Figure 19:

(a) Compaction of the subgrade developed a marked increase in subgrade bearing capacity for this soil.

(b) The increase in supporting value due to compaction was much greater for the lower 12 in. than for the top 12 in. of the compacted layer.

The average increase in bearing capacity for the lower 12 in. of compacted subgrade at Saskatoon can be represented by the equation,

$$T_{12} = 120 \log \frac{S_{12}}{S_0} \qquad (1)$$

(Fig. 19), and for the full depth of 24 in. of compacted subgrade, can be expressed by the equation

$$T_{24} = 160 \log \frac{S_{24}}{S_0} \qquad (2)$$

Where

T_{12} and T_{24} = Thicknesses of 12 in. and 24 in. of compacted subgrade respectively

S_0 = subgrade support on a 24-in. diameter bearing plate at 0.5-in. deflection for 10 repetitions on the uncompacted subgrade.

S_{12} and S_{24} = subgrade support on a 24-in. diameter bearing plate at 0.5-in. deflection for 10 repetitions on 12 in. and 24 in. of compacted subgrade respectively.

In our paper for last year's annual

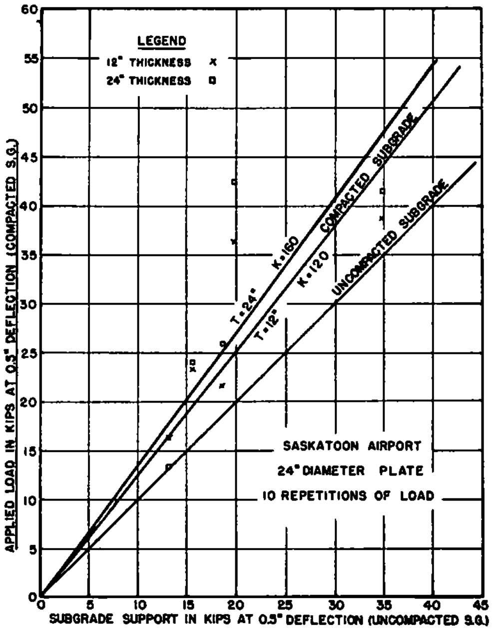

Figure 19. Influence of depth of subgrade compaction on bearing capacity

meeting *(1)* an equation was derived for expressing the supporting value provided by any thickness of well compacted granular base course material on a cohesive subgrade. The equation is

$$T = K \log \frac{P}{S} \tag{3}$$

T = thickness of granular base in inches
P = applied load
S = subgrade support at the same deflection, number of repetitions of load, and contact area that pertains to P.
K = base course constant, and has a value of 60 for a 24-in. plate (see Figure 30 below)

From a comparison of equation (3) with equations (1) and (2), it can be easily calculated that compaction of the lower 12 in. of the compacted subgrade layer at Saskatoon provided the same increase in supporting value that would have been given by a well constructed granular base 6.0 in. thick placed on the uncompacted subgrade. Similarly, a granular base course 9.0 in. thick on the uncompacted subgrade would have been required to obtain the increase in supporting value provided by compaction of the subgrade to a depth of 24 in. An economic comparison of the cost of subgrade compaction versus the cost of the greater thickness of base course required when subgrade compaction is not employed, cannot be entirely justified on this basis, however, since the value of the smaller differential settlement and the smoother pavement surface which results from compacting the subgrade, is not taken into consideration.

The increase in bearing capacity due to compaction could be expected to vary with the nature of the subgrade soil, with the degree of compaction as compared with the density of the uncompacted material, and with its topographical and geographical location. While the data contained in this section were derived experimentally for the airport at Saskatoon, the quantitative values might be different for airport sites elsewhere, and should be employed with caution unless checked by actual field or laboratory tests for the location to which they are to be applied.

6. *Influence of Subgrade Compaction on Cone Bearing, Field C.B.R., Housel Penetrometer, and Triaxial Compression Versus Plate Bearing Values*

At each test location for which subgrade plate bearing data were determined for the new runways at Saskatoon, cone bearing, field C.B.R., Housel penetrometer, and triaxial compression data were obtained for each 6-in. layer of the 2 feet of compacted subgrade, and for each 6-in. layer of the underlying uncompacted subgrade to a depth of 24 in.

By averaging the values for the four 6-in. layers of compacted subgrade, an overall average for the 2-ft. depth was obtained for each of the four tests. Similarly, overall average values for each of the four tests for four successive 6-in. layers were determined for the 24-in. depth of uncompacted subgrade. However, cone bearing values

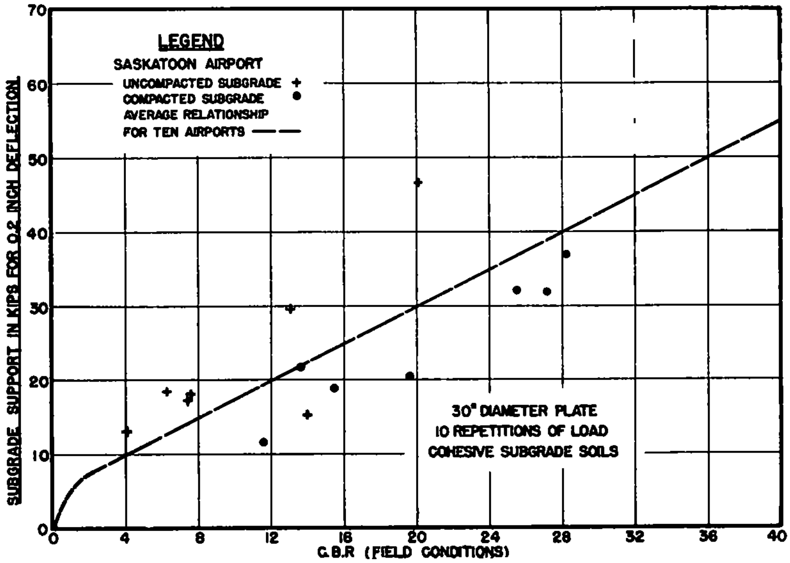

Figure 20. Subgrade support versus field CBR

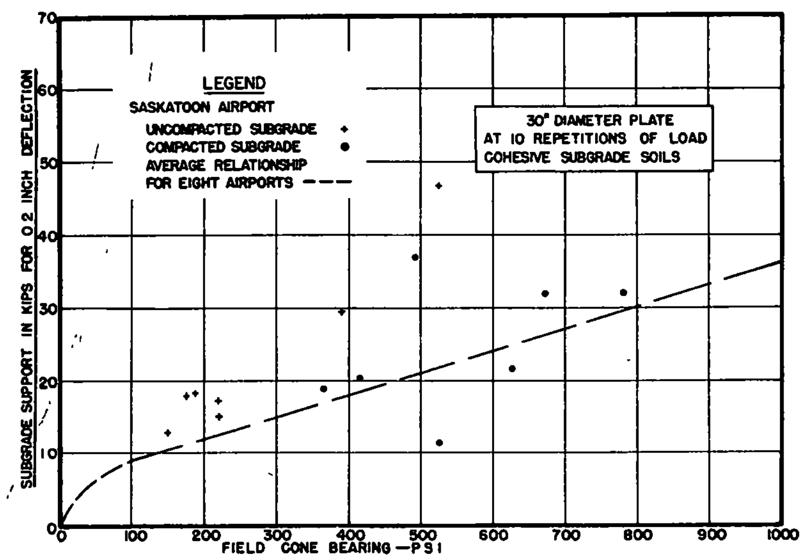

Figure 21. Subgrade support versus field cone bearing

which were higher for the first 6-in. layer than for lower layers were discarded when overall cone bearing averages were calculated. An occasional triaxial compression value for ϕ was also obviously out of line, and was deleted when determining the overall average for this test for both compacted and uncompacted portions of the subgrade.

In Figures 20, 21, 22, and 23, the field C.B.R., cone bearing, Housel penetrometer, and triaxial compression data respectively, are plotted versus the load supported on a 30-in. plate at 0.2 in. deflection for 10 repetitions of load, for both compacted and uncompacted portions

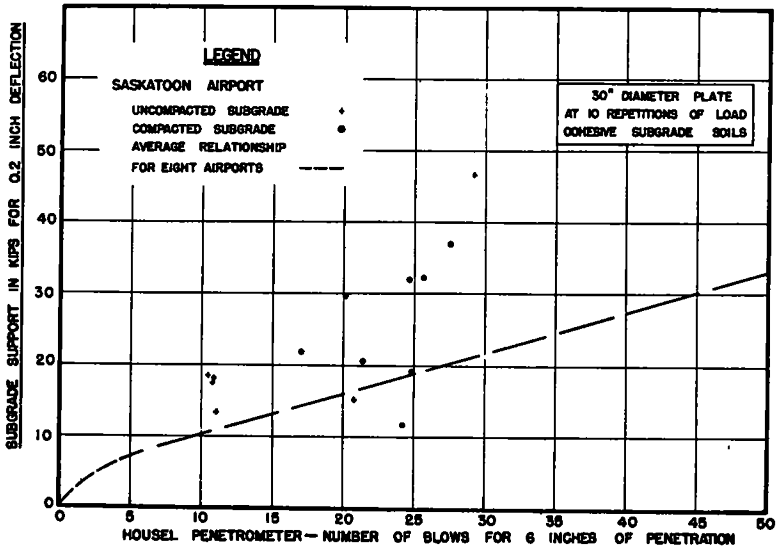

Figure 22. Subgrade support versus Housel penetrometer

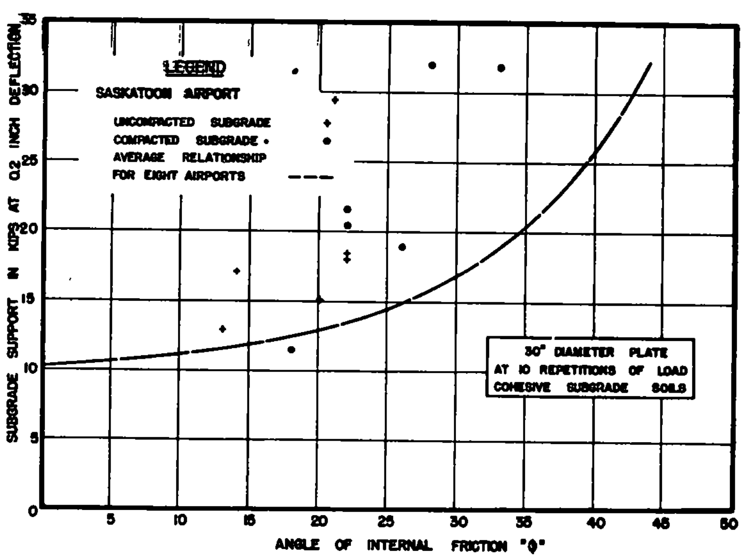

Figure 23. Subgrade support in kips at 0.2-in deflection vs. angle of internal friction

of the subgrade. The broken line curve in each of Figures 20, 21, 22, and 23, represents the average relationship between each of the four tests and the plate bearing test established previously for uncompacted subgrades (*1*), (with the exception noted in the next sentence), on the basis that if any one of the four tests were employed instead of the plate bearing test, the actual subgrade support would not be overestimated by more than 10 percent for any airport. However, it should be noted that the curves of Figures 20 and 23 have been revised somewhat from those of Figures 44 and 60 respectively, of last year's paper (*1*), as a result of study of all corresponding CBR, triaxial, and plate bearing data obtained so far. Consequently, the curves of Figures 20, 21, 22, and 23, represent the best relationships with the corresponding plate bearing values that can be established in the case of each of the four tests, on the basis of all the test data that have been obtained up to the present for subgrades that had been covered with flexible pavements for a time, and which had not been compacted to high density during construction.

centages greater than 100 show that the representative curve for that test has overestimated the actual subgrade support by the amount of the difference between the percentage given and 100 percent. Similarly, the subgrade support as given by the representative line for any one of the four tests, has been underestimated wherever the percentage shown in Table 4 is less than 100 percent.

The data of Table 4, and Figure 20, indicate that a somewhat different relationship exists between field CBR versus plate bearing tests for the compacted than for the uncompacted subgrade at Saskatoon Airport No. 2. Subgrade compaction has caused the field CBR value to increase at a considerably faster rate than the plate bearing value, since the points for the

TABLE 4

Ratio of load test values given indirectly by representative lines through data for cone bearing, Housel penetrometer, field C.B.R., and triaxial compression tests, versus actual load test values given by corresponding plate bearing tests. Data for compacted and uncompacted subgrade layers at Saskatoon Airport No. 2. Ratios expressed as percentages.

Saskatoon Airport No. 2 Subgrade Layer	Cone Bearing	Housel Penet.	Field C.B.R.	Triaxial Comp.	Overall Average For 4 Tests	Average for Cone Bearing & Housel Pen.
Compacted	102.1	84.1	127.3	65.9	96.8	93.1
Uncompacted	66.1	67.5	85.2	72.4	70.3	66.8

In table 4, a comparison is made between the load test information obtained indirectly from the representative curve through the field CBR, cone bearing, Housel penetrometer, and triaxial compression data of Figures 20, 21, 22, and 23, respectively, versus the actual load test data provided by corresponding plate bearing tests, for the compacted and uncompacted subgrade layers at Saskatoon Airport No. 2. Table 4 indicates for both compacted and uncompacted layers, the deviation between load test information obtained indirectly from the correlation curves for these four tests, versus the actual load test results determined by corresponding plate bearing tests. Per-

uncompacted subgrade tend to lie above the representative line in Figure 20, while those for the compacted subgrade are below the line. The data of Table 4 for the CBR test show that use of the representative curve of Figure 20, would lead to overestimating the supporting value of the compacted subgrade by 27.3 percent on the average, but would underestimate the supporting value of the uncompacted subgrade by 14.8 percent. Insofar as these relatively limited data are concerned therefore, the field CBR test indicates a considerably greater improvement in subgrade bearing capacity due to compaction than has actually occurred, and this could lead to serious underdesign, particularly for subgrade soils in the low CBR range.

Figure 21 and Table 4 demonstrate that a somewhat similar conclusion can be drawn with respect to the cone bearing test, but without the tendency toward serious underdesign. Use of the representative curve of Figure 21 would lead to overestimating the supporting value of the compacted subgrade by only 2.1 percent, but would underestimate the supporting value of the uncompacted subgrade by 33.9 percent.

Table 4 indicates that the representative curve of Figure 22 for the Housel penetrometer test somewhat underestimates the supporting value of both the compacted and uncompacted subgrade. The supporting value of the compacted subgrade is underestimated by 15.9 percent, and of the uncompacted subgrade by 32.5 percent.

Figure 23 illustrates the location of the data for the compacted and uncompacted subgrade layers with respect to the representative line that has been established between the angle of internal friction ϕ from the triaxial compression test versus the corresponding plate bearing values for all subgrades tested so far. Table 4 indicates that the actual subgrade supporting value is underestimated in each case by about 30 percent. Consequently, the percentage of deviation is approximately the same for both compacted and uncompacted layers, and the triaxial test is therefore the only one of these four tests for which anything approaching this degree of uniformity occurs for both the compacted and uncompacted materials.

From the right-hand column of Table 4, it can be seen that by averaging the plate bearing values obtained indirectly from cone bearing and Housel penetrometer data for both compacted and uncompacted subgrade layers by means of Figures 21, and 22, average subgrade bearing values are determined that are within 33.2 percent of the actual average plate bearing value for the uncompacted subgrade layer, and within 6.9 percent for the compacted layer. The actual subgrade support is underestimated by these percentages in each case, and is therefore on the safe side for design. From a comparison of the two right-hand columns of Table 4, it will be observed that averaging the values for these two tests, provides deviations from the actual subgrade support that are no greater, practically speaking, than the deviations obtained by averaging the values for the four tests. At 8 out of 11 airports with cohesive subgrades tested up to this time, this method, based upon averaging plate bearing values obtained indirectly from cone bearing and Housel penetrometer data, has provided results that were within 10 percent of those determined by actual plate bearing tests, and at the other three airports it has underestimated the actual subgrade support by 23, 25, and 33 percent, which is on the safe side for design. At no airport with a cohesive subgrade investigated so far, has this method led to overestimating the actual subgrade support by more than 10 percent.

On the basis of these rather limited data therefore, it appears that the actual increase in bearing capacity due to compaction of a cohesive subgrade in the field, might be obtained with reasonable accuracy from triaxial data on samples taken to the laboratory and utilizing the relationship of Figure 23, or by means of the method just outlined for averaging plate bearing data obtained indirectly from cone bearing and Housel penetrometer tests performed in the field, in conjunction with the relationships of Figures 21 and 22.

In Figure 24, values of cone bearing and angle of internal friction ϕ, are plotted versus the moisture contents and densities indicated by the corresponding modified AASHO compaction curve for a sample of soil from Saskatoon Airport No. 2. At maximum density and optimum moisture, the angle of internal friction ϕ is 37 deg., and the corresponding cone bearing value is 2300 psi. From Figure 33, it is apparent that the thickness of base course required to support any given wheel load over a subgrade with a cone bearing rating of 2300 psi, is much less than the thickness indicated when the rating of the subgrade support is given by ϕ = 37 deg. Better agreement between the thicknesses required by corresponding angles of internal friction ϕ, and cone bearing values, is obtained for the moisture contents and densities from the wetter portion of the wet branch of the compaction curve.

From the plate bearing, cone bearing, and ϕ values determined for compacted layers in the field at Saskatoon Airport No. 2, Figures 21 and 23, and from diagrams similar to Figure 24 prepared from data obtained for samples of the same subgrade layers that were compacted in the laboratory, it appears that the actual subgrade support after compaction is greatly exaggerated by the cone bearing ratings for laboratory compacted samples in the vicinity of optimum moisture and maximum density, but is indicated with reasonable accuracy by the ϕ values obtained from the triaxial compression test.

Consequently when an estimate of the probable increase in subgrade strength due to compaction is required on the basis of laboratory data, it is believed that this estimate can be made with reasonable accuracy by determining the ϕ values for the compacted and uncompacted soil by means of the triaxial compression test. The probable increase in strength due to compaction can be determined from the increase in ϕ, and from a correlation of ϕ with plate bearing data such as that shown by the curved line relationship of Figure 23. It should be unnecessary to add that when such tests are to form the basis for pavement design, both the compacted and uncompacted samples should be tested at the worst condition of moisture content expected for the subgrade after the pavement is laid.

7. *Influence of Base Course Thickness on Subgrade Modulus*

For rigid pavement design the subgrade modulus k is normally determined from the load supported on a 30-in. plate at 0.05 deflection in a static load test on the subgrade, although it was pointed out last

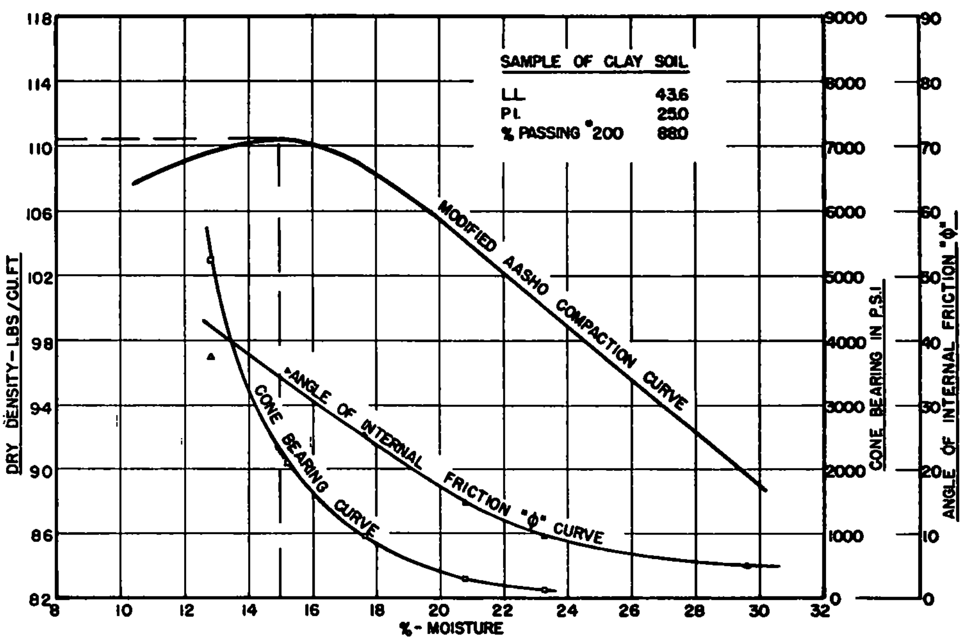

Figure 24. Influence of soil moisture and density on angle of internal friction and cone bearing values

year *(1)* that repetitive load-tests with larger bearing plates might provide more representative values for the degree of subgrade support actually available for a rigid pavement.

It is quite common practice to place a granular base course over cohesive subgrades and under the pavement. This raises the problem of calculating what the subgrade modulus k on top of any given thickness of granular base course will be, if the subgrade modulus for the subgrade itself has been measured.

In Figures 25, 26, and 27, the load carried by a 30-in. plate at 0.05 in. deflection for 10 repetitions of load on thicknesses of base course of 7, 14, and 21 in. respectively, has been plotted versus the load supported on the underlying subgrade for the same conditions of test, for a number of test locations at several airports.

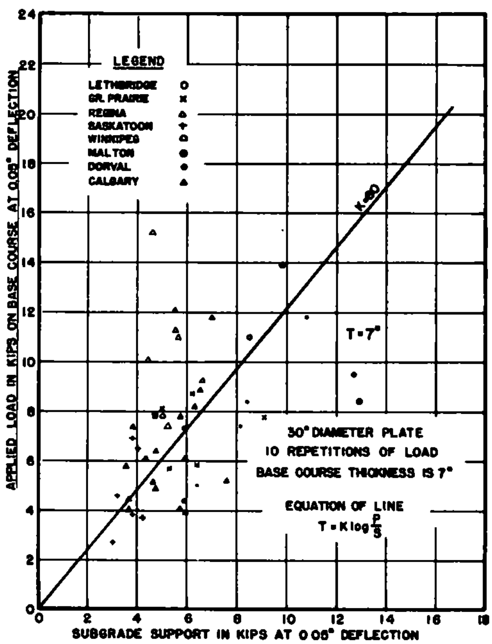

Figure 25. Applied load in kips on base course at 0.05-in. deflection versus subsupport in kips at 0.05-in deflection

It is observed that the straight line representing the equation

$$T = K \log \frac{P}{S} \qquad (3)$$

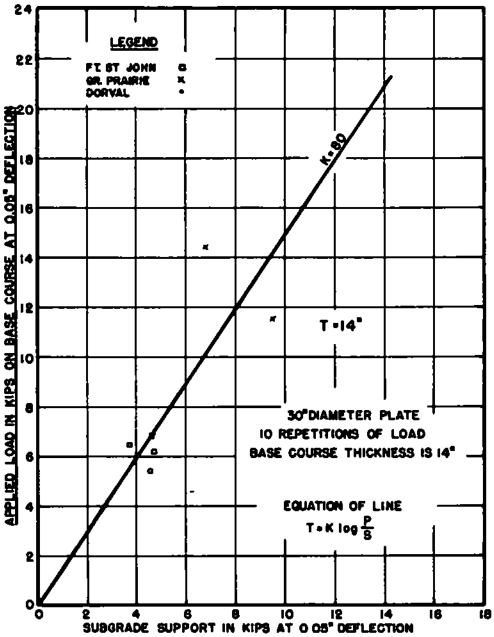

Figure 26. Applied load in kips on base course at 0.05-in deflection versus subgrade support in kips at 0.05-in. deflection

expresses the relationship between the data reasonably well, where each symbol has the significance previously explained for equation (3).

Therefore, if the subgrade modulus k_s for the subgrade has been measured, the subgrade modulus k_b at the surface of a well compacted granular base course of any thickness T can be calculated from the equation

$$T = K \log \frac{k_b}{k_s} \qquad (4)$$

While the best average value of K for the 30-in. plate for flexible pavement design appears to be 65 (1), Figures 25, 26 and 27, indicate that K = 80 might be more representative for calculating the subgrade modulus k_b for a granular base course for rigid pavement design, by means of equation (4). Actually, insofar as its influence on the required thickness of a rigid pavement is concerned, it matters very little what value of K in equation (4) over the range of 65 to 80, is taken, as the following example illustrates.

If the subgrade modulus k_s for a given project is 100 lbs./cu. in., and it is covered with a 12-in. thickness of well compacted granular base, it is found by substitution in equation (4), that k_b, the value of the subgrade modulus at the surface of the base course is 153 lbs./cu. in. if K = 65, and is 142 lbs./cu. in. if K = 80. Insofar as rigid pavement design for most wheel loads is concerned, a subgrade modulus of either 141 or 153 lbs./cu. in. would result in the same pavement thickness, practically speaking.

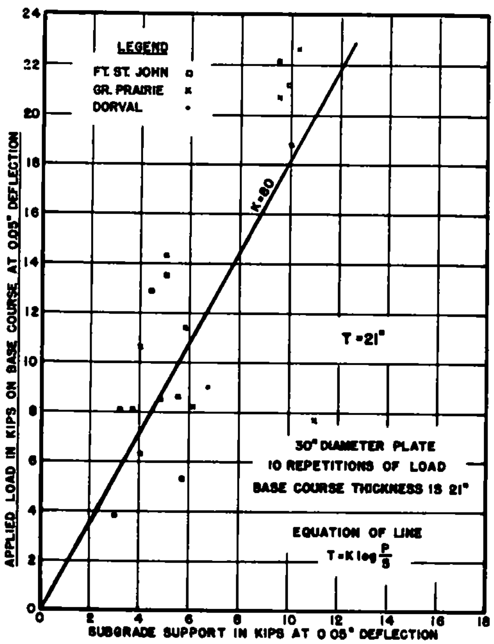

Figure 27. Applied load in kips on base course at 0.05-in. deflection versus subgrade support in kips at 0.05-in. deflection

Consequently, if the subgrade modulus k_s for the subgrade has been measured, the subgrade modulus k_b at the surface of any thickness T of well compacted granular base course can be calculated by means of the equation.

$$T = 80 \log \frac{k_b}{k_s} \qquad (5).$$

If a loose poorly compacted granular base course is employed, its supporting value for a rigid pavement may be less than that of an underlying cohesive subgrade. It cannot be too strongly emphasized therefore, that equation (5) will apply only for granular base courses that have been thoroughly compacted. The degree of compaction should be not less than 100 percent of modified AASHO maximum at any point.

8. *Supporting Value of Bituminous Surface Relative Granular Base*

For bituminous pavements that have been in service for sometime, the load test data have demonstrated that 1-in. of bituminous surfacing made with liquid asphalt or soft asphalt cement has the supporting value of 1.5 in. of granular base, and that this ratio is 2.5 for well designed and constructed penetration macadam, sheet asphalt and asphaltic concrete (*1*).

Load test data obtained during the 1947 testing program, have indicated on the other hand, that 1-in. of a newly constructed bituminous surface may have a somewhat smaller supporting value than 1-in. of granular base.

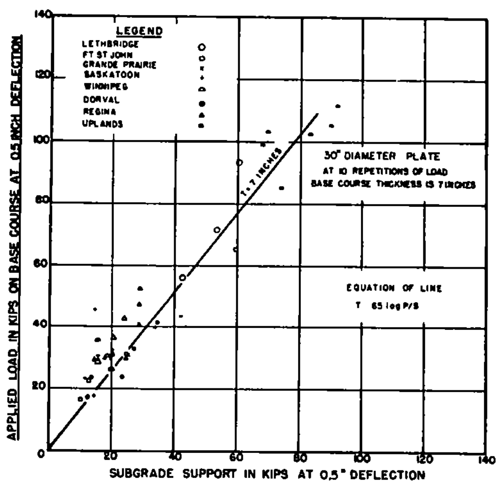

Figure 28. Applied load in kips on base course at 0.5-in. deflection versus subgrade support in kips at 0.5-in. deflection

9. *Flexible Pavement Design Equation for Sandy Subgrades*

Figure 28 is a graph of applied load versus subgrade support, both determined with a 30-in. plate, 0.5 in. deflection, 10 repetitions of load, for a granular base course 7 in. thick.

Data for Uplands airport at Ottawa, where the subgrade consists of about 80 feet of relatively clean sand, are seen to be represented quite well by the best line through the data for other airports, where the subgrade consists of cohesive soil. In the absence of comprehensive data from a number of airports with sandy subgrades, it would appear from Figure 28, that flexible pavement design equation (3)

$$T = K \log \frac{P}{S} \qquad (3)$$

can be tentatively employed to determine the required thickness of granular base courses over subgrades of sandy soils.

10. *Influence of Bearing Plate Size on Value of the Base Course Constant K*

For the paper presented last year (*1*), it was found that the best average value for the base course constant K in equation (3) for a 30-in. plate, was 65. It was also believed at that time that the value of K was independent of the size of the bearing plate.

From data obtained during 1947, and from closer scrutiny of the earlier data, it has been found that the base course constant K varies with the size of bearing plate. Figure 29 demonstrates that the average value of K for a 12-in. plate is about 35.

From a study of all available data, it appears that the change in value of K with change in size of bearing plate can be expressed as a straight line relationship when K is plotted versus P/A ratio, Figure 30.

11. *Thickness Design Charts for Flexible Pavements for Airports*

Figures 94, 95, 96, and 97 of last year's paper (*1*), for required thickness of granular base for runways were derived from equation (3), $T = K \log (P/S)$, and were based upon 0.5 in. deflection for 10 repetitions of load, and upon a value of $K = 65$ which was independent of size of contact area. Similarly, the

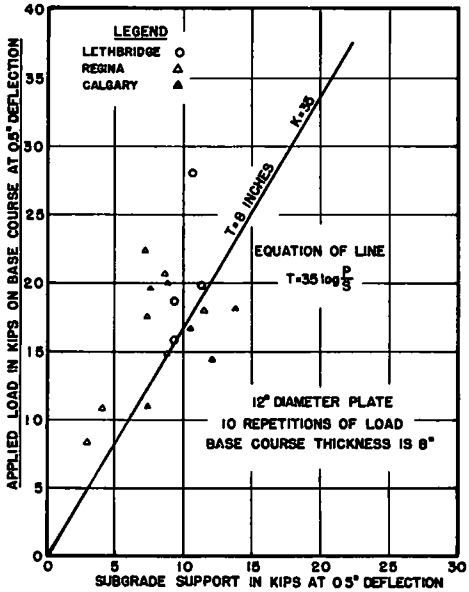

Figure 29. Applied load in kips on base course at 0.5-in. deflection versus subgrade support in kips at 0.5-in. deflection

thicknesses for taxiways, aprons and turn-arounds, as given by Figures 96 and 97, were also obtained from equation (3), based upon $K = 65$ and 0.225 in. deflection for 10 repetitions of load. This approach resulted in an almost constant difference of about 10 to 11 in. between the thickness requirements for runways and for taxiways, etc., regardless of the magnitude of the wheel load. It would be more reasonable to expect that this difference would increase with an increase in wheel load, and vice versa.

Figures 31, 32, and 33, are revised charts which indicate the thicknesses of granular base required for runways, and for taxiways, aprons, and turnarounds, for a wide range of airplane wheel loads for capacity operations. They are based upon flexible pavement design equation (3), and upon a deflection of 0.5 in. for 10 repetitions of load for runways, and upon a deflection of 0.35 in. for 10 repetitions of load for taxiways, etc., but the value of K varies with the size

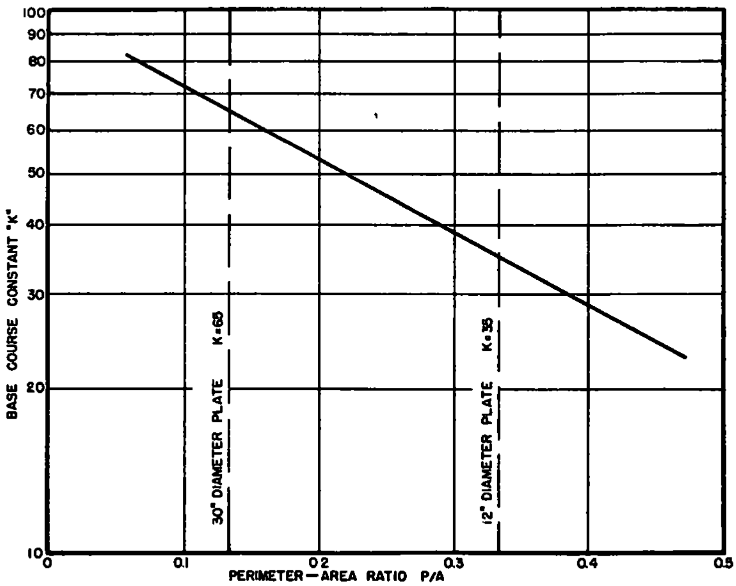

Figure 30. Influence of perimeter - area ratio on value of *K* in flexible pavement design equation

of contact area as indicated by Figure 30. This results in a difference between thickness requirements for runways, and for taxiways, etc., which changes with the magnitude of the wheel load, Figures 32 and 33. This difference varies from about 3 in. for a 5000 lb. wheel load to about 6 in. for a wheel load of 150,000 lb.

Thickness design for runways for capacity operations is based upon 0.5 in. deflection for 10 repetitions of load, since this has been indicated by correlating load test with traffic data. For taxiways, aprons, and turnarounds, the design criterion should be the yield point deflection to avoid progressive settlement of parked aircraft into the pavement. Figure 7 indicates that the average yield point deflection for the flexible pavements for the thirteen airports tested so far, occurs at 0.35 in. Consequently, the load which develops 0.35 in. deflection for 10 repetitions has been selected as the design criterion for taxiways, aprons, and turnarounds for capacity operations.

In Figure 31, the runway thickness requirements of the U.S. Corps of Engineers for subgrade soils with soaked CBR values of 3.0 and 4.5, are shown on the curves by circles and crosses, respectively (5). The cross-hatched portion of Figure 31 outlines the range of thicknesses indicated by equation (3) based upon actual load test data (0.5 in. deflection for 10 repetitions) for eight Canadian airports at which the average CBR ratings of soaked samples varies from 2.2 to 4.6. The cross-hatched portion of Figure 31, which represents the results of combined traffic and load test information obtained during the Department of Transport's investigation, demonstrates that very satisfactory service performance has been obtained from runways with thicknesses of base and surface that vary from one-third to four-fifths of those required by U.S. Corps of Engineers' design. This difference in thickness requirements was commented upon at length in last year's paper (*1*). It is due to the fact that the Department of Transport's investigation has demonstrated that the moisture contents of the top inch of the soaked subgrade samples, upon which the CBR test that forms the basis for the Corps of Engineers' design is made, were

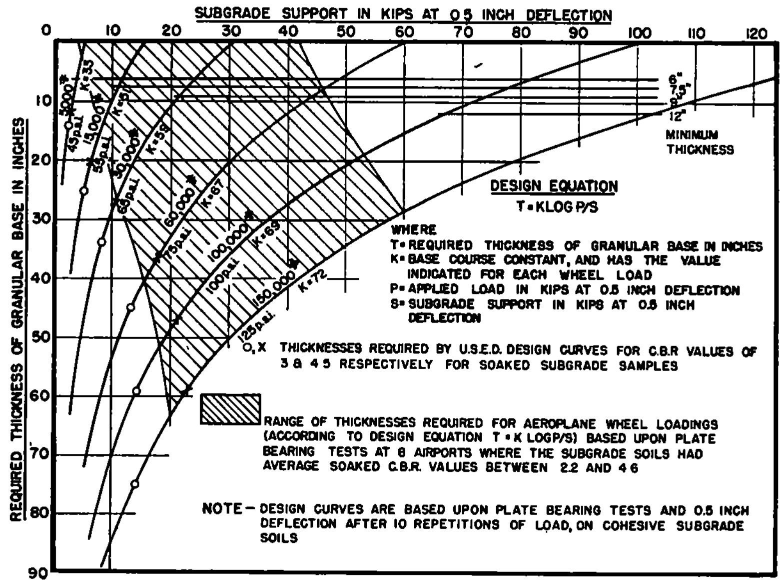

Figure 31. Design curves for flexible pavements for airplane wheel loadings (full load on single tire)

from 25 to 140 percent higher than the actual moisture contents of the subgrades under the pavements at the Canadian airports investigated so far.

Canadian airport engineers have been interested to observe that their own experience concerning flexible pavement design for runways, in comparison with what is considered to be the ultra conservative design of the Corps of Engineers, has been quite adequately confirmed by Palmer (6) recently, with regard to a considerable number of airports in the United States which are being currently investigated by the Bureau of Yards and Docks of the U. S. Navy.

Although they are considerably smaller than those of the Corps of Engineers, it is not unlikely that the thickness requirements for runways as given in Figures 32, and 33, for capacity operations, are still too conservative. At Dorval airport near Montreal for example, there were up to 40 operations per day by Constellations and D.C.4's for some weeks during the Spring and summer of 1947, and the runways have supported several tens of thousands of operations with wheel loads of 30,000 pounds or more, without sign of distress. By U.S.A. standards however, this might be classified as limited traffic for this wheel load. The Department of Transport study has indicated that the criterion of runway design for capacity operations should be 0.5 in. deflection for 10 repetitions of load, and for limited traffic should be 0.5 in. deflection for 1 repetition of load.

For the contact area of a 30,000 lb. wheel load, the subgrade support for the two weakest runways at Dorval is 11,300 lbs. at 0.5 in. deflection for 10 repetitions of load, and is 13,000 lbs. at 0.5 in. deflection for 1 repetition of load. For this subgrade support, equation (3) indicates that the thickness requirement for a runway for a 30,000 lb. wheel load for capacity operations is

$$T = 60 \log \frac{30,000}{11,300}$$
$$= 25.5 \text{ in.}$$

and for limited operations is

$$T = 60 \log \frac{30,000}{13,000}$$
$$= 22 \text{ in.}$$

Therefore, according to the design for thickness of granular base for flexible pavement developed by the Department of Transport's investigation, if the traffic at Dorval is considered to be in the "limited" category, the required thickness is 22 in. If traffic is in the "capacity" class, the thickness required is 25.5 in.

equation (3), and the actual thickness (in terms of equivalent thickness of granular base) approximate each other, even on the basis of limited operations. In addition, it is to be remembered that Dorval has been subjected to considerable traffic by aircraft with wheel loads greater than 30,000 lbs. Furthermore, at nearly all Canadian airports, the runways serve also as taxiways, and this is particularly true of Dorval. Figures 32 and 33, indicate that for a wheel load of

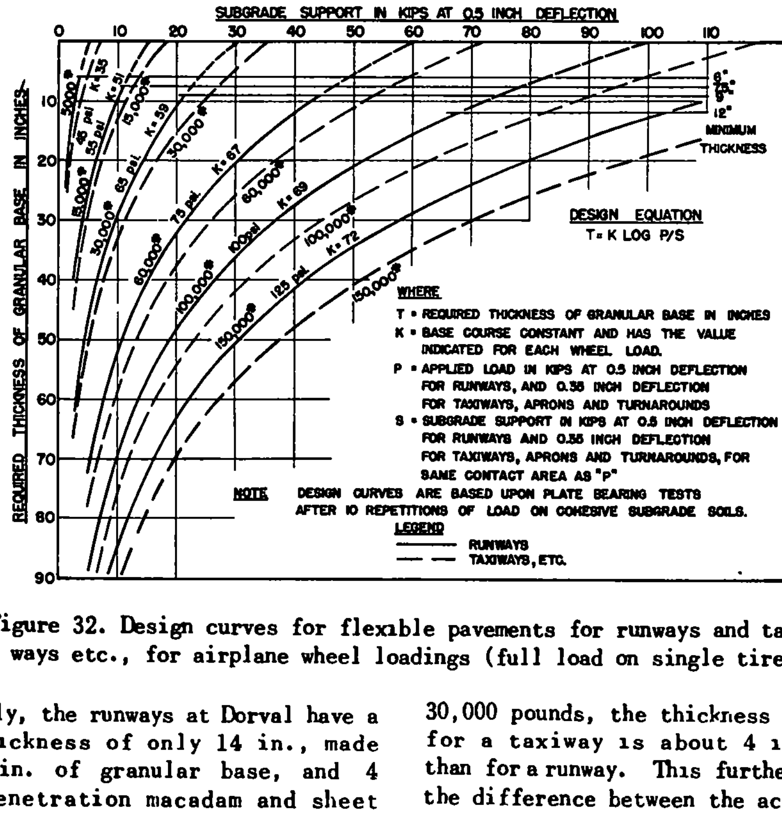

Figure 32. Design curves for flexible pavements for runways and taxiways etc., for airplane wheel loadings (full load on single tire)

Actually, the runways at Dorval have a total thickness of only 14 in., made up of 10 in. of granular base, and 4 in. of penetration macadam and sheet asphalt. If each inch of bituminous surface at Dorval has the supporting value of 2.5 in. of granular base, as conservatively indicated by plate bearing tests, the base and pavement at Dorval are equivalent in supporting value to 20 in. of granular base.

Consequently, it is only by allowing for the greater supporting capacity of the bituminous surface per inch of thickness, as compared with granular base, that the design thickness as required by

30,000 pounds, the thickness requirement for a taxiway is about 4 in. greater than for a runway. This further increases the difference between the actual thickness of the Dorval runways, and the thickness requirement indicated by equation (3), illustrated earlier in this section. Therefore, as far as the runways at Dorval are concerned, flexible pavement design equation (3), $T = K \log (P/S)$, and the design charts of Figures 32, and 33, provide for conservative thicknesses of granular base, rather than otherwise. It may be that further field testing and traffic studies will indicate that smaller thicknesses than these could be safely

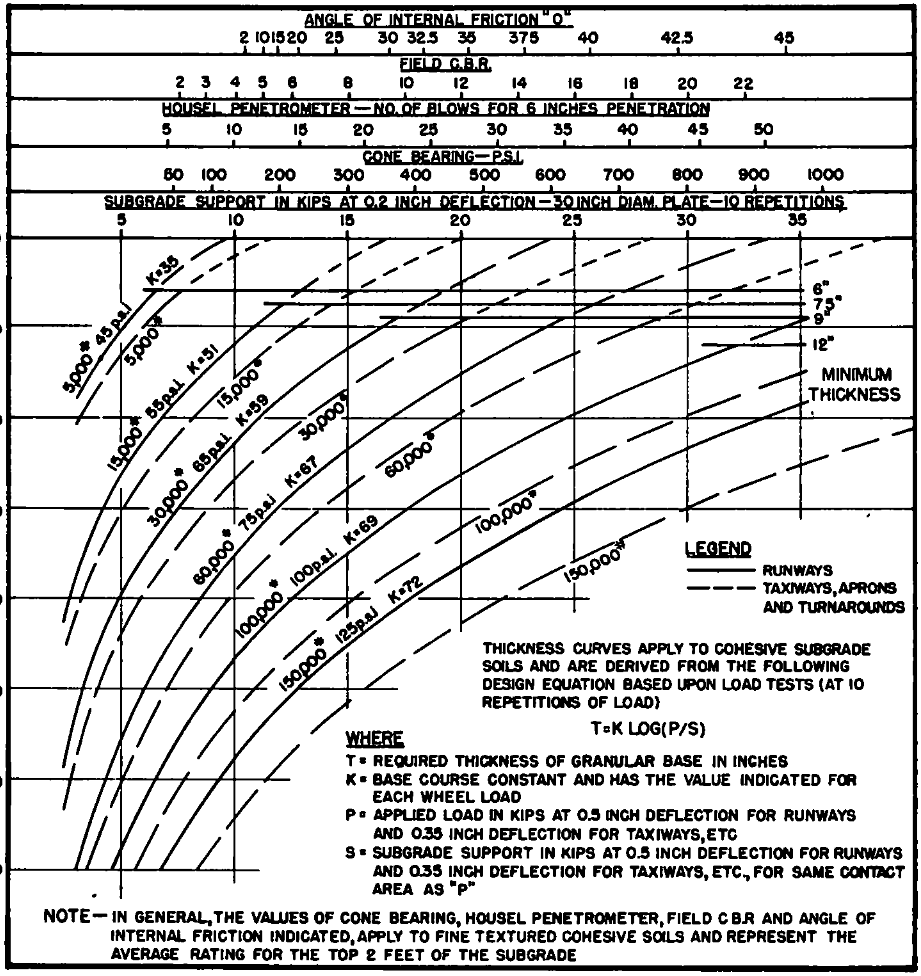

Figure 33. Design curves for flexible pavements for runways and taxiways, etc., for airplane wheel loadings (full load on single tire) (Based upon five subgrade tests)

employed for runway design in general.

It should be particularly noted that the curves in Figures 31, 32, and 33, are for the full load on a single tire, and that the thickness of granular base required is considerably less when the load is carried on dual tires. Most commercial aircraft with wheel loads greater than 30,000 pounds are equipped with dual tires at the present time. From data obtained by the Department of Transport for dual and single steel bearing plates of the same contact area (1), it appears that for any given thickness of granular base and flexible surface up to about 20 in., the load supported on dual tires is from one-third to one-half greater than that which can be carried on a single tire. For thicknesses greater than about 20 in., this ratio probably begins to decrease. For aircraft equipped with dual tires therefore, the thickness requirements would be appreciably less than those shown in Figures 31, 32, and 33.

11. Thickness Design Charts for Flexible Pavements for Highways

Figures 99 and 100 of last year's paper (*1*), contained curves for thickness requirements for flexible pavements for highway wheel loadings, which were based upon equation (3), a deflection of 0.5 inch for 10 repetitions, and upon a value of $K = 65$ which was independent of the size of contact area. Figure 30, however, indicates that $K = 35$ is the value of the base course constant K for a 12-in. plate, which approximates the contact area for a heavy highway wheel load. Using $K = 35$ in place of $K = 65$, with all other factors remaining equal, would cut the thickness requirements of Figures 99, and 100 (1) approximately in half, which would undoubtedly be inadequate, and a somewhat different approach than that employed a year ago is indicated.

deflection. The Department of Transport's investigation has indicated that 0.35 in. for 10 repetitions, is the average yield point deflection for flexible pavements studied so far. Therefore, the revised curves of thickness requirements for highway wheel loadings illustrated in Figures 34, and 35, are based upon flexible pavement design equation (3), for a deflection of 0.35 in. for 10 repetitions of load, and upon a value of K which varies with size of contact area, as indicated by Figure 30. The curves of thickness requirements given by Figures 34 and 35, appear to be reasonable for the various wheel loadings, for a high density of traffic.

The significance of the small circles and crosses on the four curves, and of the cross-hatched area of Figure 34, was commented upon at length in last year's paper (*1*). The distance between the circle and cross on each curve of Figure 34 represents the small range of thick-

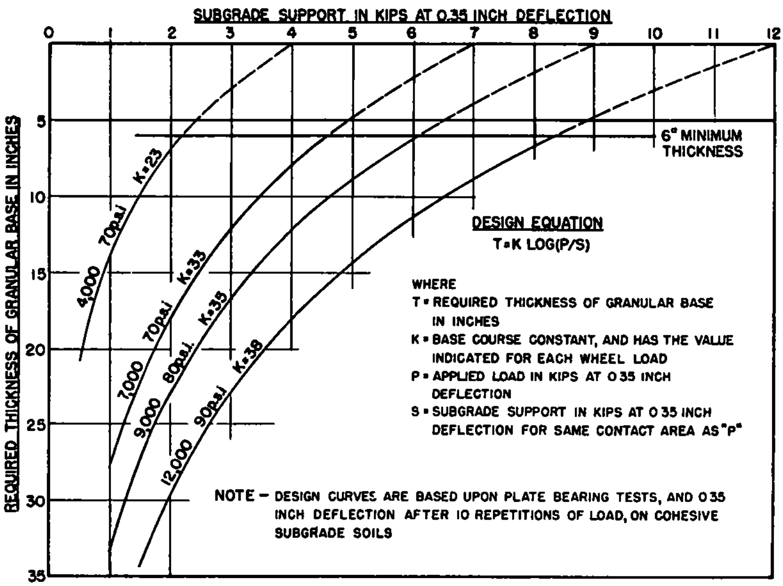

Figure 34. Design curves for flexible pavements for highway wheel (full load on single tire)

To avoid settlement into the road surface when vehicles are parked or stalled, flexible pavements for highways should probably be designed for the yield point

nesses of flexible pavements for highway wheel loadings permitted by the Corps of Engineers' design chart (5), and presumably by the California Division of Highways (*7*), for subgrades having soaked CBR ratings from 3.0 to 4.5. Actual plate bearing tests at Canadian airports with subgrades having soaked CBR values between 2.2 and 4.6, have warrented the very much wider range of thicknesses shown by the shaded portion of Figure 34, because the actual moisture contents of these subgrades are considerably less than those of the thoroughly soaked samples upon which the standard CBR test is made.

load. For an exceptionally high intensity of traffic, design might be based upon 0.35 in. deflection for 100 repetitions of load. Table 5 of last year's paper (*1*) indicated that for the same contact area, and for any given deflection over the range of 0.2 to 0.7 in., the loads carried at 1 repetition and 100 repetitions were 115 and 89 percent respectively, of the load supported at 10 repetitions.

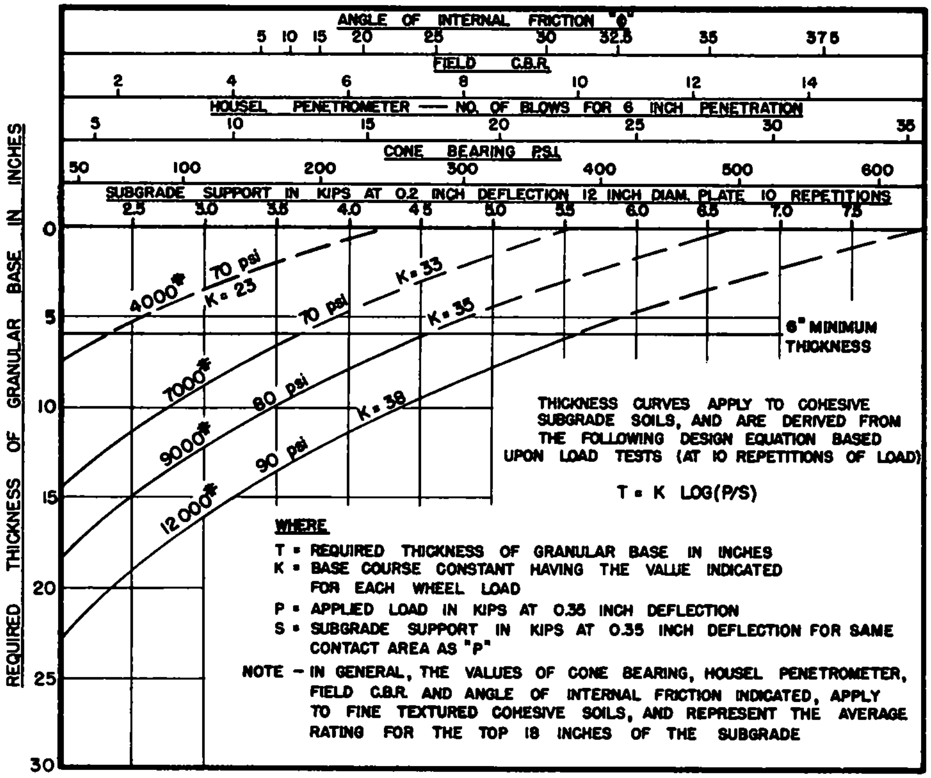

Figure 35. Design curves for highway wheel loadings on cohesive subgrade soils (full load on single tire)

When employing equation (3) for a high density fo highway traffic, flexible pavement design should be based upon 0.35 in. deflection for 10 repetitions of load. Where the anticipated traffic density is low, the design criterion should be 0.35 in. for 1 repetition of

12. *Flexible Pavement Design Equation and the Angle of Spread*

From flexible pavement design equation (3), $T = K \log (P/S)$, the angles of spread for the transmission of pressure through different thicknesses of granular base course can be calculated, on the assumption that the subgrade support is uniform across the base of each cone of pressure.

The product of a uniform applied unit load p at the surface multiplied by the contact area (assumed circular) over which it is acting, must be equal to the product of a uniform unit subgrade support s_o on any horizontal plane below, multiplied by the circular area over which it is distributed, Figures 37 and 38.

Subgrade support s in equation (3) refers to the same contact area as p. However, it is obvious that when p and s are unequal as they usually are, the products obtained when each is multiplied by the same contact area will be unequal. That is, subgrade support s must be multiplied by a larger contact area if the product is to be equal to that of the applied load p multiplied by the surface contact area over which p is acting. However, unit subgrade support s diminishes for any given deflection as the contact area is increased over which it is applied, in accordance with the straight line relationship of unit load versus P/A ratio, Figure 4. The change in unit subgrade support s with size of contact area, is such that the unit load on a 12-in. bearing plate is approximately twice that on a 30-in. plate for any given deflection over the range of 0.2 to 0.5 in.

Keeping all these factors in mind, it is a relatively simple matter to calculate the radius of the circular area over which the unit subgrade support s_o must be uniformily distributed, so that the product of s_o multiplied by this circular area will be equal to the product of the applied unit load p multiplied by the contact area at the surface over which it is acting, for each thickness T of granular base course. From the radii of these circular areas and the corresponding thicknesses of base course, the angles of spread for different depths of base course can be determined. Explanatory diagrams, the necessary equations for a 30-in. bearing plate, and a sample calculation are outlined in Figure 36.

Figure 37 illustrates the results of this analysis when it is applied to the flexible pavement design equation $T = 65 \log(p/s)$ for a 30-in. diameter bearing plate on which the unit applied load is 85 psi. This corresponds to an airplane wheel load of approximately 60,000 pounds. The table in Figure 37, indicates that the angle of spread increases fairly rapidly with depth of base course for this design equation.

Figure 38 provides similar information for the flexible pavement design equation, $T = 35 \log(p/s)$, applied to a 12-in. diameter plate on which the unit load is 80 psi. This approximates a highway wheel load of 9,000 pounds. It is again apparent, from the table in Figure 38, that for this design equation, the angle of spread increases rather rapidly with thickness of base course.

Both Figures 37 and 38 indicate that in general, the angle of spread determined from equation (3) is a great deal less than 45 deg., the value upon which several formulae for determining the required thickness of flexible pavement is based.

It is of interest that for a thickness of very nearly 13 inches, the table in Figure 38 indicates that the angle of spread has the value of 24 deg. 54 min., which Bail (*8*) reports is employed for flexible pavement design by the New Mexico State Highway Department. For a base course thickness of very nearly 16 in., the table in Figure 38 shows that the use of the equation, $T = 35 \log(p/s)$, provides an angle of spread of 26.5 deg., which according to Bail (*8*) is utilized by the Texas State Highway Department for flexible pavement design.

For both Figures 37 and 38, the angles of spread given by equation (3), $T = K \log(p/s)$, may increase too rapidly for the greater thicknesses of base course. A smaller rate of increase of angle of spread with thickness of base would be obtained, if the base course constant K were considered to increase with depth of granular base in accordance with equations (23) or (27) of last year's paper (*1*).

13. *The Selection of Base Course Materials and the Design of Bituminous Mixtures*

This topic was touched upon in the paper for last year's meeting in a rather limited manner (*1*). Further study has led to the more complete development which is outlined below.

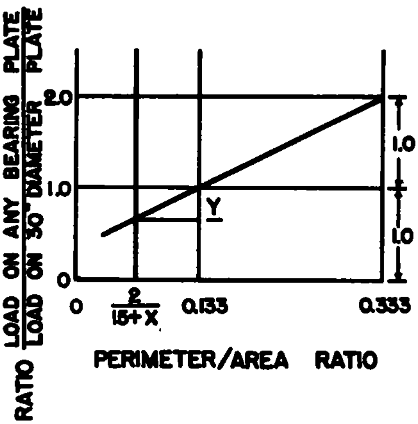

DIAGRAM A

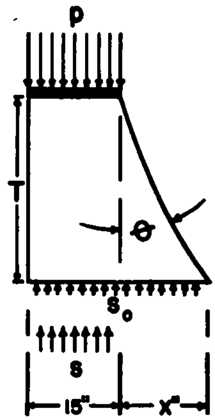

DIAGRAM B

FLEXIBLE PAVEMENT DESIGN EQUATION $T = 65 \log p/s$

FOR GIVEN VALUES OF "p" AND "T"

$$\log s = \log p - \frac{T}{65}$$

$$s = \text{antilog}\left(\log p - \frac{T}{65}\right)$$

TO DETERMINE ANGLE OF SPREAD "θ"

FROM DIAGRAM A

$$\frac{Y}{0.133 - \frac{2}{15+X}} = \frac{1}{0.333 - 0.133}$$

$$Y = \frac{0.667\ X}{15 + X}$$

FROM DIAGRAMS A AND B

$$s_o = 1 - Y$$

$$s\left(1 - \frac{0.667X}{15+X}\right)\pi(15+X)^2 = p\pi\ 15^2$$

$$s(15 + 0.333\ X)(15+X) = 225\ p$$

$$0.333X^2 + 20\ X + 225 = \frac{225\ p}{s}$$

$$X^2 + 60\ X + 675 - \frac{675p}{s} = 0$$

$$X = \frac{-60 + \sqrt{60^2 - 4\left(675 - \frac{675\ p}{s}\right)}}{2}$$

$$\tan\theta = \frac{X}{T}$$

SAMPLE CALCULATION

IF $p = 85$ p.s.i.

AND $T = 15$ INCHES

$$\log s = \log 85 - \frac{15}{65}$$

$$s = 49.96$$

$$X = \frac{-60 + \sqrt{60^2 - 4\left[675 - \left(\frac{675 \times 85}{49.96}\right)\right]}}{2}$$

$$X = 7.058$$

$$\tan\theta = \frac{7.058}{15} = 0.4705$$

$$\theta = 25^\circ\ 12'$$

Figure 36. Derivation of angle of spread given by flexible pavement design equation (airplane wheel loadings)

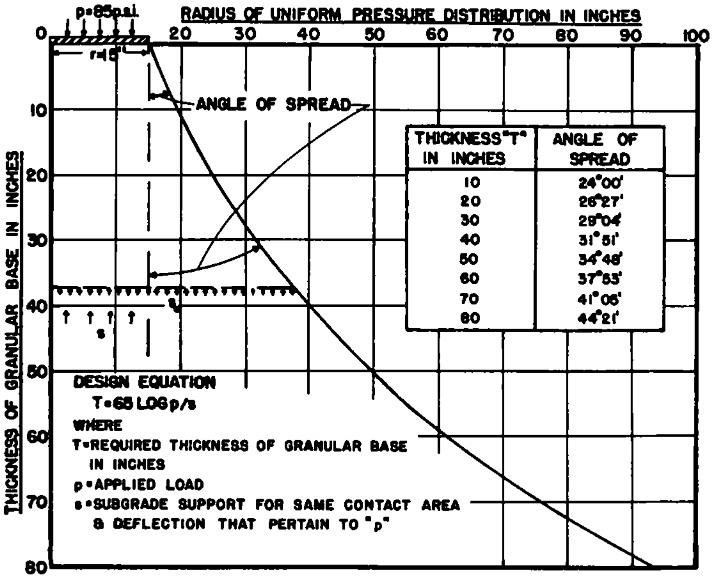

Figure 37. Angle of spread of uniform pressure distribution given by the flexible pavement design equation (airplane wheel loadings)

While the heading of this section would ordinarily be considered to deal with two quite separate subjects, this is not true for the approach to be presented here, wherein the design of bituminous mixtures (insofar as stability is concerned) is indicated to be merely a limited case of the more general development applicable to the selection and design of base course materials.

Figure 39 is a diagram of possible planes of shearing failure under a loaded area on the surface of a flexible pavement on an airport or highway. The principal problem of design consists of preventing detrimental shear in the subgrade, base course, and wearing surface. If sufficient plastic shear develops in any one or more of these three elements, rutting and upheaval of the surface will occur.

Detrimental plastic shear of the subgrade is prevented by an adequate overlying thickness of base course and pavement, as given by equation (3). However, the base course material and the flexible surface must also each possess the required stability (resistance to plastic shear) under the imposed stress.

It is a serious drawback to the design of bituminous pavements and granular base courses at the present time, that it cannot be placed on the basis of pounds per square inch, in terms of flexural strength, sheer, or similar property, as is the case for the design of steel columns or girders, rigid pavements, etc. In addition, there is the difficult problem of finding a common basis for comparing the stabilities of the various base course materials available for any project, and the stabilities of bituminous mixtures made from the different aggregate materials at hand, in order that the most economical selection of each may be made. At the present time, some engineers object to the use of soil-bituminous mixtures for base courses, even for locations where they should be favored for economic reasons because granular materials are not locally available. This objection is due very largely to the lack of a common basis for comparing the stabilities of the granular and soil-bituminous types of base course materials.

can provide a fundamentally sound basis of comparison for the selection of base course and aggregate materials insofar as their stability is concerned, and that it can be utilized for designing the stability or strength of flexible bases

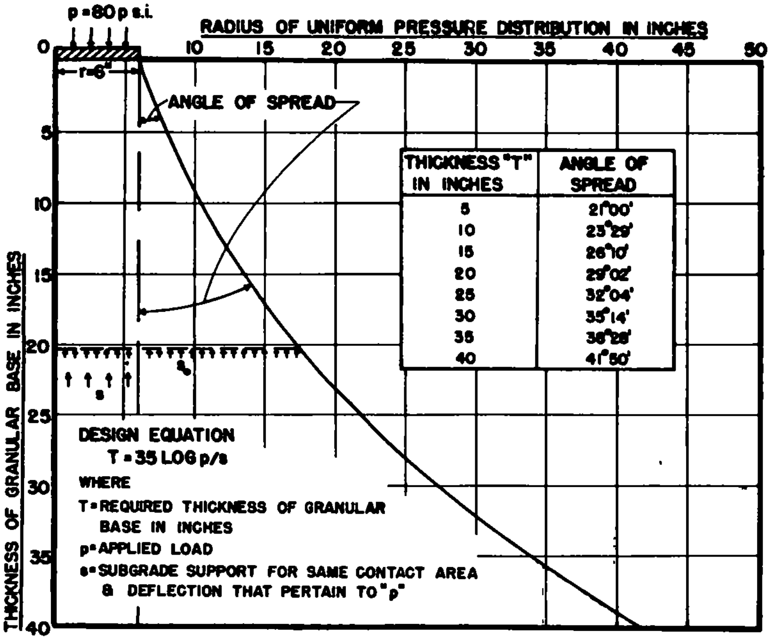

Figure 38. Angle of spread of uniform pressure distribution given by the flexible pavement design equation (highway wheel loadings)

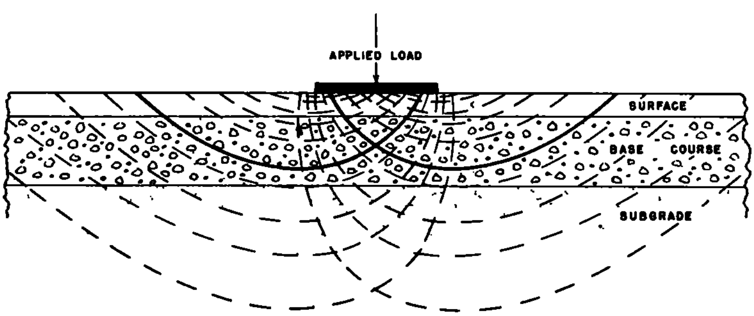

Figure 39. Diagram of shear planes under a loaded area

The development which follows, indicates that the triaxial compression test and wearing surfaces on a psi basis.

A triaxial compression test differs

from an ordinary compression test, in that provision is made for controlled lateral support while the specimen is subjected to vertical load. Figure 40 is a diagram of the essential equipment for this test. To the lucite cylinder, the two metal end pieces are fitted by means of water-tight and air-tight gasketed joints. A cylindrical specimen of the material to be tested is inserted in a rubber sleeve between porous stones at top and bottom.

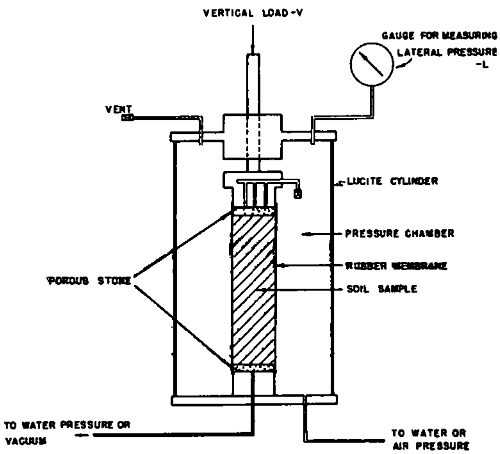

Figure 40. Sketch of apparatus for triaxial compression test

By means of connections through the porous stones, the material within the rubber sleeve can be subjected to either vacuum or water pressure, if desired. Water or air can be pumped into the lucite cylinder to provide the magnitude of lateral support required when testing each specimen. The rubber sleeve prevents water within the lucite cylinder from entering the sample. Each specimen is subjected to a constant lateral pressure throughout the test, and increasing vertical load is applied in a standard manner until it fails. A complete triaxial compression test usually consists of loading three or four cylindrical specimens of a given material to failure, employing a different degree of lateral support for each specimen, e.g. 0, 15, 30 and 60, psi.

The data obtained from testing a given material in triaxial compression are plotted in the form of a Mohr diagram, Figure 41. The applied lateral pressure L, and the corresponding vertical pressure V which caused failure, are marked off on the horizontal axis for each test specimen. Using the difference between the vertical and lateral pressure, $V - L$, for each specimen as the diameter, semi-circles, known as Mohr circles, are described, The tangent which is common to the Mohr circles is drawn and produced to intersect the vertical axis. The intercept made on the vertical axis is designated cohesion c, from the Coulomb equation $s = c + n \tan \phi$, while the angle between the common tangent and the horizontal is the angle of internal friction ϕ.

The common tangent is generally known as the Mohr rupture line or Mohr envelope. All semi-circles which are tangent to or below the Mohr envelope, represent equilibrium or stable relationships respectively, between corresponding values of lateral pressure L and vertical pressure V. Any semi-circle which cuts through the Mohr envelope, indicates corresponding combinations of lateral pressure L and vertical pressure V which would cause failure of the material being tested.

For the development which follows, it is assumed that the Mohr envelope is a straight line. This assumption appears to be justified on the basis of recent published reports by Holtz (*9*), Rutledge (*10*), Nijboer (*11*), and others.

The Mohr diagram provides a fundamental basis for defining the term "stability" as applied to granular and cohesive materials in general, and to flexible base course and surfacing materials in particular. Granular and cohesive materials may be classified in terms of increasing stability according to their capacity for carrying a greater applied vertical load V for a given value of lateral support L. Consequently, for the equilibrium conditions of stress established by the Mohr envelopes for a number of materials under comparison, for any specified value of lateral support L the most stable material is that for which the value of $(V - L)$ is greatest.

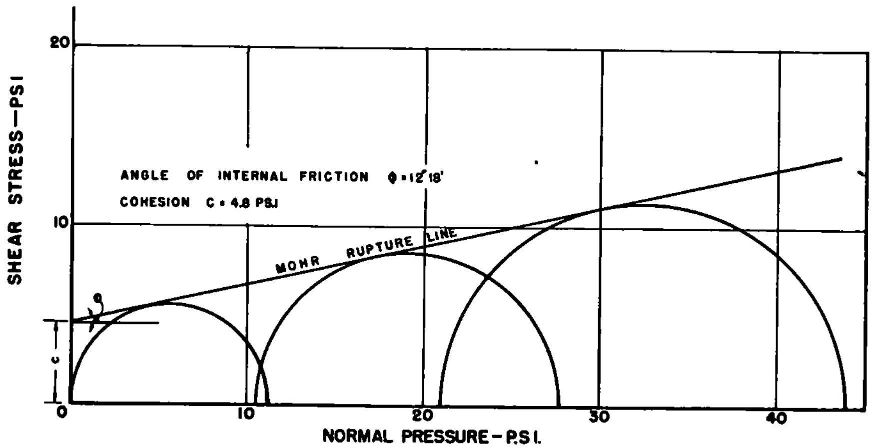

Figure 41. Typical Mohr diagram for triaxial compression test

While the terms, vertical load V, and lateral support L, as they are frequently designated for the triaxial test, will be employed throughout this paper, it is to be understood that they have the significance, in the widest sense, of major and minor principal stresses respectively, which are usually denoted by σ_1 and σ_{111}.

By reference to the Mohr diagram, cohesive and granular materials can be conventionally divided into three groups:

(a) purely cohesive materials, i. e. those having a positive value for cohesion c, but for which the angle of internal friction ϕ is zero, Figure 42. Saturated clays in the quick triaxial test approximate these requirements.

(b) purely granular materials, i. e. those having a positive value for the angle of internal friction ϕ, but for which the cohesion c is zero, Figure 44. Clean sands and gravels approach this condition.

(c) materials which have both granular and cohesive properties, i. e. those having positive values for both c and ϕ, Figure 41. Bituminous paving mixtures, mechanically stabilized base courses with positive values for plasticity index, and remolded clays, are common examples.

a. The Stability of Purely Cohesive Materials - Figure 42 illustrates the Mohr diagram for a purely cohesive material. The angle of internal friction ϕ is zero. The Mohr envelope is parallel to the abscissa, and at a distance c from it. Regardless of the magnitude of the lateral support L, the diameter of any Mohr circle, that is, $(V - L)$, which represents the stability of the material, is a constant for any specified value of c.

The mathematical equation of stability for a purely cohesive material is, therefore,

$$V - L = 2c \tag{6}$$

and the stability diagram for a purely cohesive material, based upon this equation, is shown in Figure 43.

If a vertical load V of 150 psi were to be supported by a base course which could develop a maximum lateral support L of 50 psi, equation (6) and Figure 43 indicate that a purely cohesive material with cohesion c equal to 50 psi minimum, would be required to provide the necessary stability.

b. The Stability of Purely Granular Materials - From Figure 44, a Mohr diagram for purely granular materials, it is apparent that the stability value $(V - L)$ depends upon the magnitude of the lateral support L, and the size of the angle of internal friction ϕ.

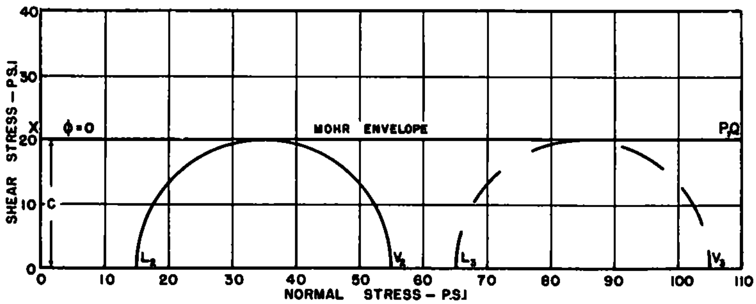

Figure 42. Mohr diagram for materials having zero angle of internal friction in triaxial compression

From a study of the trigonometrical relationships of Figure 45, it follows that,

$$r = \frac{L \sin \phi}{1 - \sin \phi} \tag{7}$$

and

$$V - L = 2r \tag{8}$$

Therefore,

$$V - L = \frac{2L \sin \phi}{1 - \sin \phi} \tag{9}$$

Equation (9) is a mathematical equation for the stability, $(V - L)$, of purely granular materials. When equation (9) is plotted in terms of $(V - L)$ curves for different values of lateral support L, and of angle of internal friction ϕ, the stability diagram of Figure 46 is obtained.

If a base course is to carry a vertical load V of 150 psi, and can develop a maximum lateral support L of 50 psi, then its stability requirement, $(V - L)$, is 100 psi. Either equation (9) or Figure 46 indicates that a purely granular material with an angle of internal friction ϕ = 30 deg. or greater, would be required to provide the necessary stability for this base course.

c. The Stability of Materials With Combined Granular and Cohesive Properties - From Figure 41, it is clear that the stability, $(V - L)$, of materials with both granular and cohesive properties, depends upon the magnitude of the lateral support L, the cohesion c, and the angle of internal friction ϕ. The geometrical

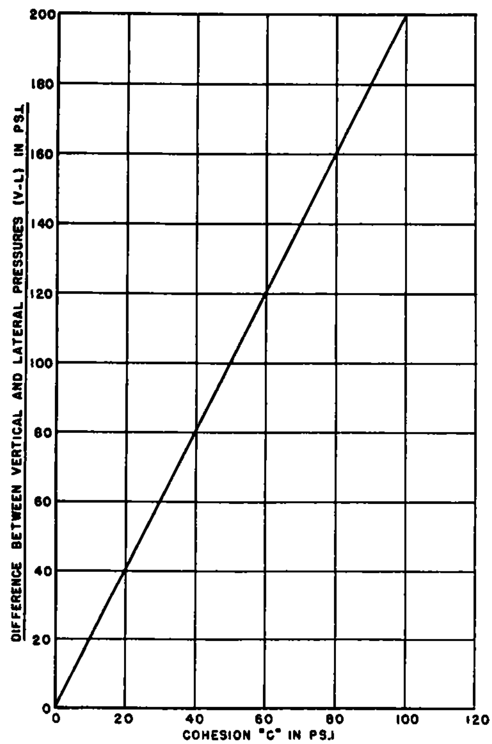

Figure 43. Stability diagram in terms of $(V - L)$ and cohesion c for materials having zero angle of internal friction in triaxial compression

and trigonometrical relationships required for the development of the equation of stability for these materials,

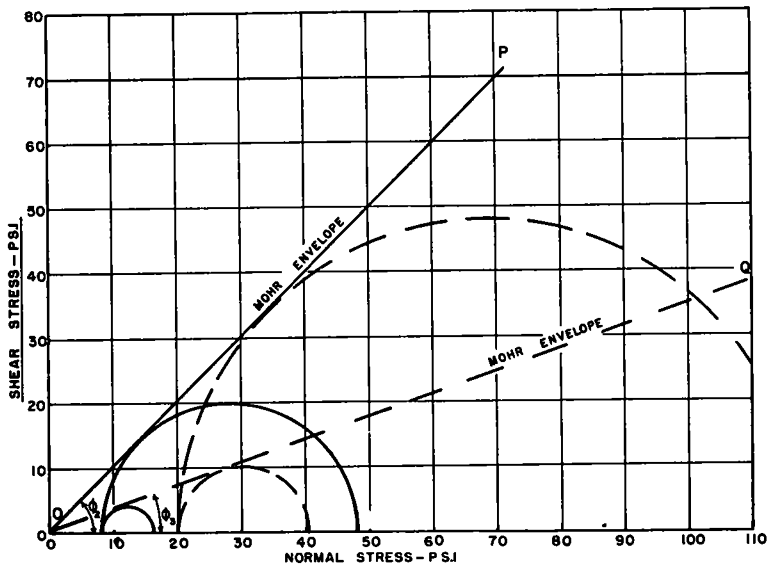

Figure 44. Mohr diagram for materials having zero cohesion in triaxial compression

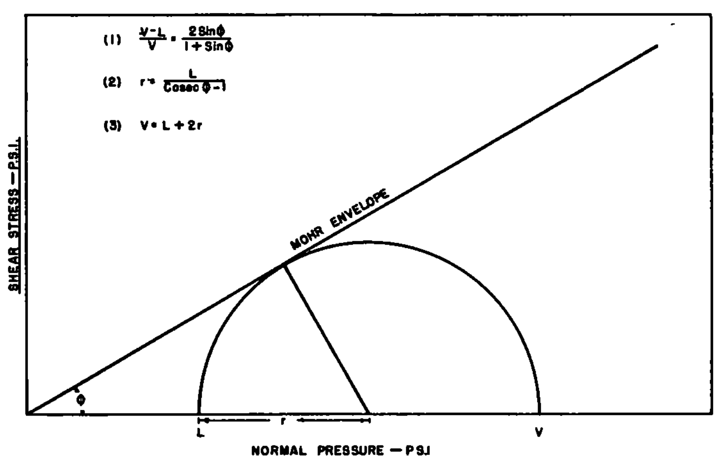

Figure 45. Trigonometrical relationships for Mohr diagram for materials with zero cohesion

are illustrated in Figure 47.

It follows from Figure 47, that,

$$\tan \phi = \frac{\frac{V - L}{2} \cos \phi - c}{L + \frac{V - L}{2} - \frac{V - L}{2} \sin \phi} \quad (10)$$

which can be worked through to

$$V - L = \frac{2\,L \sin \phi}{1 - \sin \phi} + 2\,c \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \quad (11)$$

The stability diagram of Figure 48 is obtained when equation (11) is plotted in terms of given values of stability, $(V - L)$, for different degrees of lateral support L, and for various magnitudes of c and ϕ. The $(V - L)$ stability curves of this diagram are not straight lines, although for the intermediate and higher values of lateral support L they are very nearly so. For a lateral support $L = 0$, the stability curves are concave upwards throughout, while for L equal to any value greater than zero, they are reverse curves.

The stability diagram of Figure 48 may shown as points 1, 2, 3, 4, and 5 on the curved line graph of Figure 48, representing $(V - L) = 100$ psi, and $L = 40$ psi That is, points 1, 2, 3, 4, end 5, represent Mohr envelopes AT, BT, CT, DT, and ET of Figure 49, where T is at the point of tangency for each Mohr envelope drawn to the Mohr circle in question.

For the curved line representing any one of the infinite combinations of $(V - L)$ and L values that are possible in Figure 48, there are very simple equations

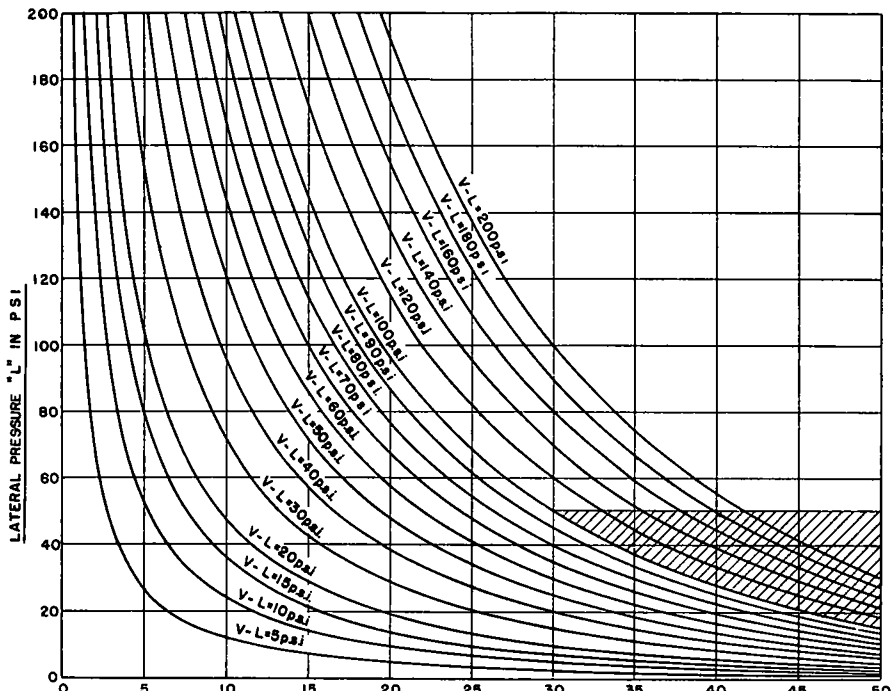

Figure 46. Stability diagram in terms of L, ϕ and $(V - L)$ for materials having zero cohesion in triaxial compression

be more readily understood with reference to the Mohr diagram of Figure 49, which contains several Mohr circles of the same diameter, that is, same $(V - L)$ value, but with different degrees of lateral support L. To one of these $(V - L) = 100$ psi, and $L = 40$ psi, several Mohr envelopes have been drawn for some of the infinite combinations of c and ϕ that are possible for this particular Mohr circle. The combinations of c and ϕ corresponding to each of these Mohr envelopes, are for locating the extremities of the line on the c and ϕ axis.

When $\phi = 0$, equation (11) reduces to equation (6)

$$V - L = 2c \tag{6}$$

and this locates the required extremity of the line on the c axis.

When $c = 0$, the extremity of this line on the ϕ axis can be obtained from the equation

$$\sin \phi = \frac{V - L}{V + L} \tag{12}$$

Referring again to the problem of designing a base course to carry a vertical load V of 150 psi, and for which the maximum lateral support L that can be developed is 50 psi, the solution can be calculated by means of equation (11), and is given graphically in Figure 40, which indicates that an infinite number of answers are possible. All materials posessing those combinations of c and ϕ which are on or to the right of the curve labelled $(V - L) = 100$ psi, $L = 50$ psi, in Figure 50, would have the required stability. Materials with those combinations of c and ϕ which lie within the cross-hatched area to the left of this line would tend to be unstable and therefore unsatisfactory, insofar as this particular base course design problem is concerned.

As deposits of good granular material for base courses become depleted, highway and airport engineers are being forced more and more to contemplate the utilization of what have been considered inferior materials. The primary requirement of a base course material is adequate stability under load. By testing inferior gravels, sands, or other materials in triaxial compression, and plotting the location of their corresponding c and ϕ values on a stability diagram like that of Figure 48, their deficiencies become immediately apparent. The problem with regard to any given inferior material is then largely one of economics, as to whether its ϕ value should be improved by incorporating a good granular material, or its c value improved by adding a suitable binder such as clay, bitumen, portland cement, etc., or whether both binder and granular material are to be incorporated. If the deficient material can be improved to the extent that under the worst conditions expected for it in the field, its combination of ϕ and c values will be located to the right of the required $(V - L)$ stability curve for the maximum lateral support L developed by the base course on that project, e.g. Figure 50, it will function as satisfactorily as the most carefully selected crushed aggregrate, insofar as base course stability is concerned.

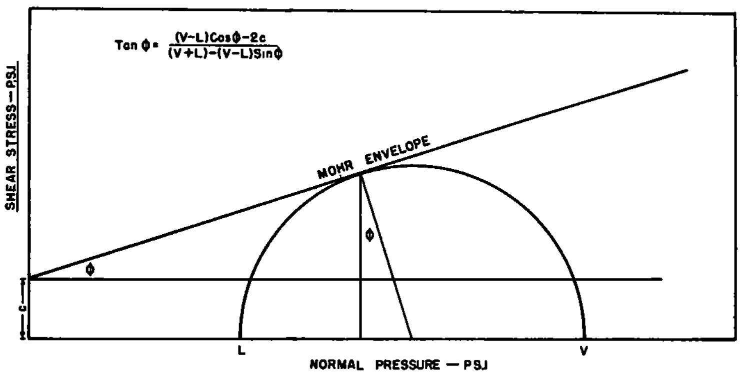

Figure 47. Trigonometrical relationships for Mohr diagram for materials having positive values of c and ϕ in triaxial compression

It might be added that this application of the triaxial test would place the design of soil-bituminous mixtures on a sound fundamental basis. Soil bituminous mixtures possess both cohesion and internal friction. By means of the development just outlined, the stabilities of soil-bituminous mixtures could be determined and compared directly with those for granular base course materials on a pounds per square inch basis.

d. The Design of Bituminous Mixtures - Regardless of the magnitude of their stability as determined by the triaxial compression test, it is common knowledge that gravel road surfaces which contain no binder of any kind, develop washboard

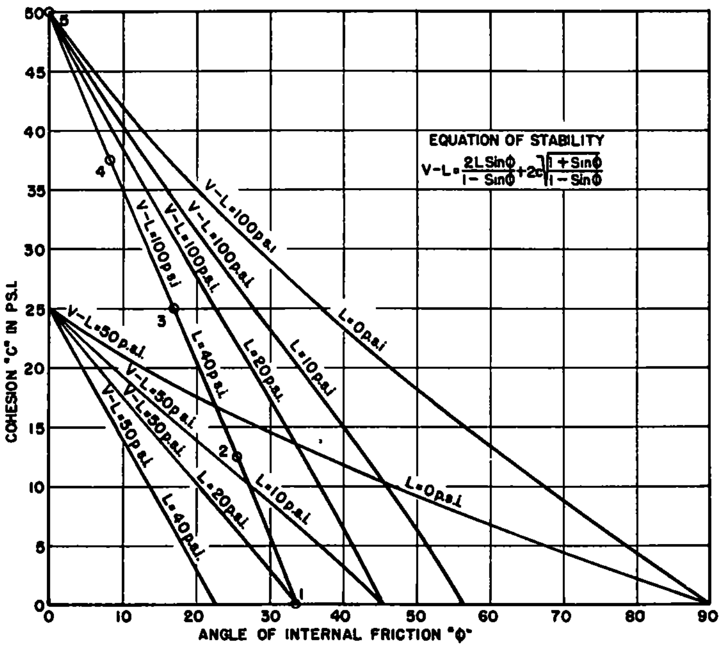

Figure 48. Stability diagram in terms of c, ϕ, L and $(V - L)$ for materials having positive values for c and ϕ in triaxial compression

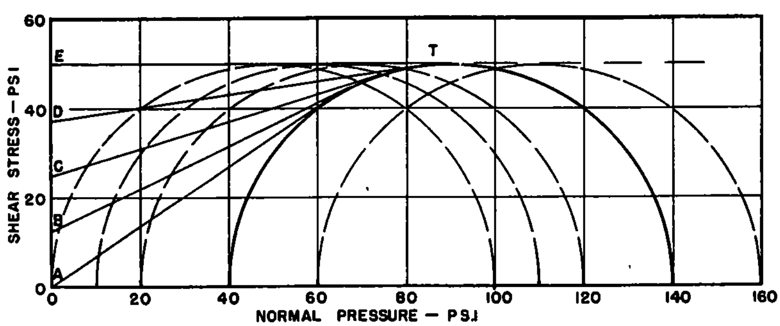

Figure 49. Mohr diagram for constant $(V - L)$ but varying lateral support L

and other indications of instability under the particular types of stresses to which the surface layer is subjected by motor vehicle and airplane traffic. Experience has shown that for satisfactory performance, the surface layer of a highway or airport must contain a binding material to give it cohesion. The most commonly employed binders are clay, bitumen, and portland cement, and should probably include moisture.

Equation (11) and the stability diagram of Figure 48 are not entirely satisfactory for the design of the surfacing for a highway or airport, since they would permit the use of materials with

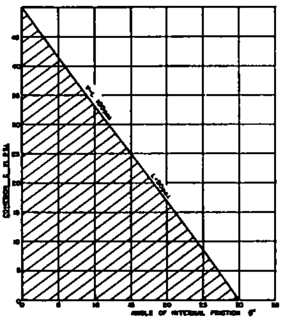

Figure 50. Stability diagram in terms of c, ϕ, L and $(V - L)$ for materials having positive values for c and ϕ in triaxial compression

even zero cohesion. The problem therefore, is to establish the minimum value of the cohesion c which is required for surfacing materials, such as bituminous mixtures, and mechanically stabilized mixtures of aggregate and soil binders.

The required minimum value of c might be determined empirically. It happens however, that there is an approach to this problem, based upon the properties of the Mohr diagram, which provides minimum values for cohesion c that are in reasonable agreement with the results of experimental studies contained in a diagram in a recent publication of the Asphalt Institute.

The geometrical and trigonometrical relationships required for this approach to the problem are illustrated in Figure 51.

When $\log \frac{V - L}{V}$ is plotted versus $\log L$ for the Mohr envelope for any material possessing both cohesive and granular properties, the reverse curve graph of Figure 52 is obtained. The value of the lateral pressure L at which the point of inflection in Figure 52 occurs, is obtained by equating the second derivative of the equation for the curve to zero. It should be noted that a reverse curve is also obtained when either $\frac{V - L}{V}$ or $\frac{L}{V}$ is plotted versus $\log L$.

The equations and mathematical derivations involved in obtaining expressions for the value of the slope of this reverse curve at any point, and of the lateral pressure L at the point of inflection, are outlined in Figure 52.

The term "L_i" is applied to the particular value of the lateral pressure L at the point of inflection of the curve in Figure 52, and the corresponding vertical pressure V is represented by "V_i". For each Mohr envelope, Figure 41, there can be only one value of L_i and one value of V_i. That is, for any one combination of values of cohesion c and angle of internal friction ϕ, there can be only one value of L_i and its corresponding value of V_i. Consequently, the corresponding values of L_i and V_i are a characteristic of each Mohr envelope.

The expressions for L_i and V_i and $(V_i - L_i)$, when reduced to their simplest forms in terms of c and ϕ are as follows,-

$$L_i = 2\,c\,\sqrt{\frac{1 - \sin\phi}{2\sin\phi}} \qquad (13)$$

$$V_i = 2\,c\;\frac{1 + \sin\phi + \sqrt{2\sin\phi}\,\sqrt{1 + \sin\phi}}{\sqrt{2\sin\phi}\;\sqrt{1 - \sin\phi}} \qquad (14)$$

$$(V_i - L_i) = 2\,c\;\frac{\sqrt{1 + \sin\phi} + \sqrt{2\sin\phi}}{\sqrt{1 - \sin\phi}} \qquad (15)$$

also

$$(V_i - L_i) = L_i\;\frac{2\sin\phi + \sqrt{2\sin\phi}\;\sqrt{1 + \sin\phi}}{1 - \sin\phi} \qquad (16)$$

In Figure 53, Mohr circles representing corresponding values of L_i and V_i have been drawn to the Mohr envelopes for c equal to unity in each case, but with values of internal friction ϕ varying from 1 deg. to 50 deg. In Figure 54, Mohr circles in terms of L_i and V_i are drawn to the Mohr envelopes for $\phi = 30$ deg. in each case, but with values of c equal to 10, 20, and 30 psi.

Figure 53 demonstrates that for a constant value of c, the value of $(V_i - L_i)$ increases as ϕ increases, and vice versa.

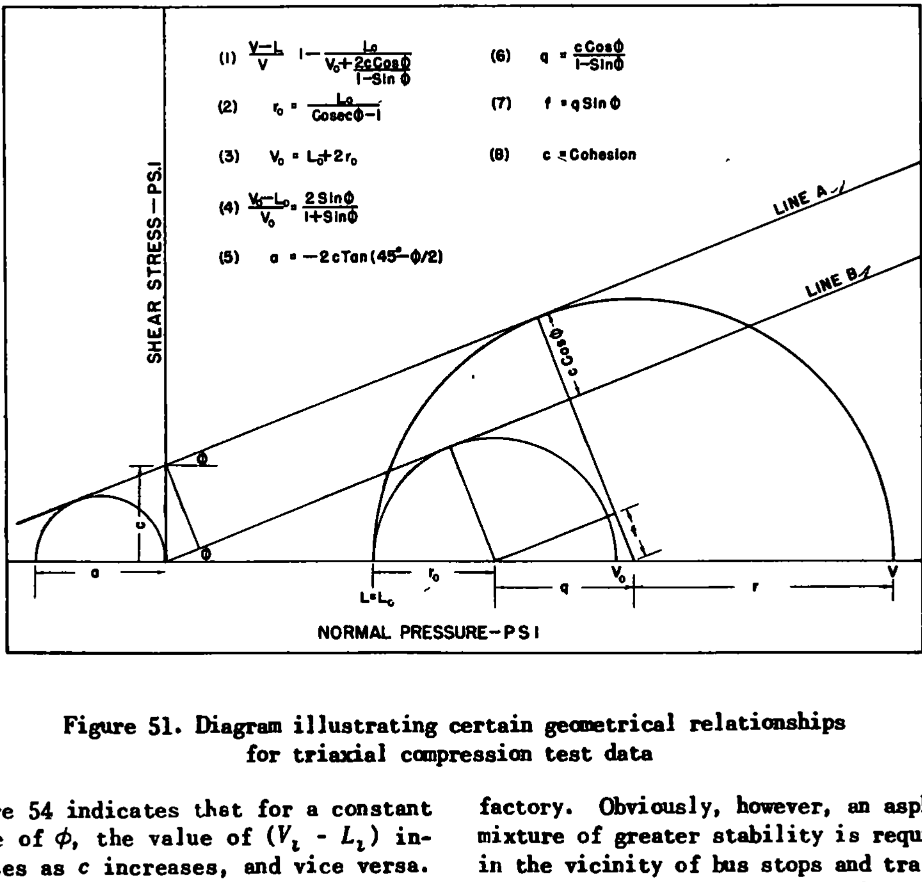

Figure 51. Diagram illustrating certain geometrical relationships for triaxial compression test data

Figure 54 indicates that for a constant value of ϕ, the value of $(V_l - L_l)$ increases as c increases, and vice versa. It has long been known that for a given lateral pressure L, the stability or strength of materials, $(V - L)$, increases, as either c or ϕ or both increase. Consequently, for any specified magnitude of L_l, the value of $(V_l - L_l)$ provides a measure of the stability of a material with combined granular and cohesive properties, (Fig. 59).

Figures 55 and 56 are graphs of different values of L_l and $(V_l - L_l)$ respectively, in terms of c and ϕ.

Figure 57 is a stability diagram for hot mix asphaltic concrete paving mixtures, based upon the triaxial compression test, which appears in the Asphalt Institute's recent manual (*12*). It will be observed that a single boundary appears between mixtures having combinations of c and ϕ labelled satisfactory and unsatisfactory. Obviously, however, an asphalt mixture of greater stability is required in the vicinity of bus stops and traffic lights, then for average traffic conditions. The diagram of Figure 57 does not indicate the combinations of c and ϕ required for increased or decreased pavement stability. It is clear therefore, that the utility of this diagram would be materially increased if it could be zoned into areas of greater or less stability.

In Figure 58 $(V_l - L_l)$ curves are superimposed upon the Asphalt Institute diagram of Figure 57. It should be noted that the curve representing a $(V_l - L_l)$ value of 80 psi coincides quite well with the lower boundary for satisfactory mixtures shown in the Asphalt Institute diagram, although better agreement would probably be obtained with the curve for a $(V_l - L_l)$ value of 70 psi. For locations such as bus stops or traffic lights, where high stability is required, bituminous mixtures might be specified that have corresponding values of c and ϕ which result in a $(V_i + L_i)$ value of 120 psi or higher. For average conditions, bituminous mixtures having combinations of c and ϕ which result in a $(V_i - L_i)$ value of 80 psi, might be satisfactory. maximum lateral support L which it can develop is 50 psi, the required stability $(V - L)$ of the paving mixture is 100 psi. The solution to this problem of paving mixture design is indicated graphically in Figure 60. Only those bituminous mixtures having combinations of c

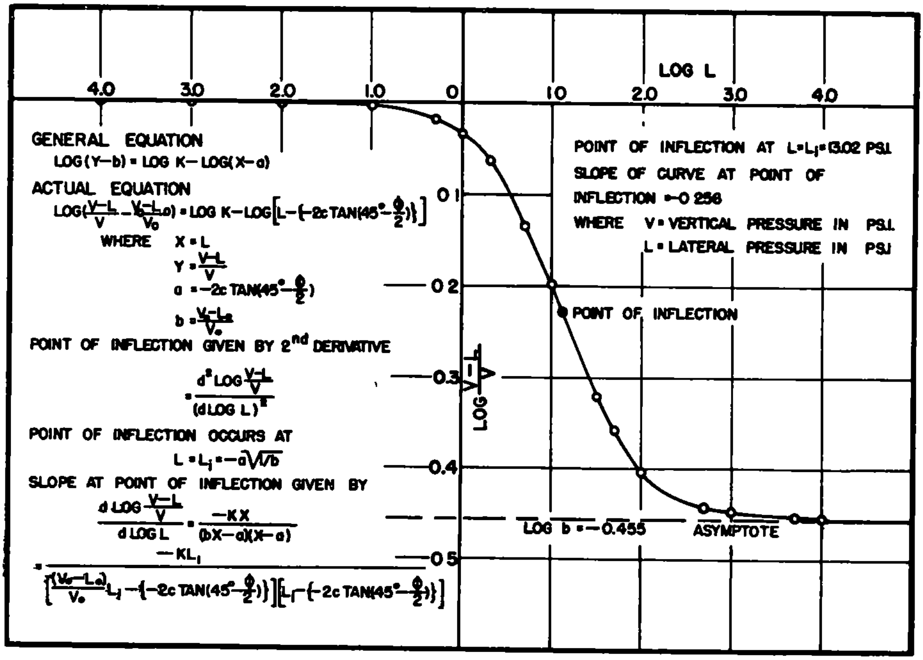

Figure 52. Log of ratio $\frac{V - L}{V}$ versus log L for triaxial compression test

The $(V_i - L_i)$ values required by bituminous pavements for different traffic conditions, could be determined by investigations in which field performance was correlated with triaxial tests on representative pavement samples.

In Figure 59, values of L_i, the lateral support at the point of inflection, from Figure 55, have been superimposed upon the $(V_i - L_i)$ curves of Figure 58. There is a maximum lateral support L, which each bituminous pavement can develop in service, and the different possible values of this lateral support L, for corresponding $(V_i - L_i)$ curves, are indicated by the L_i curves of Figure 59.

If a bituminous pavement must support a vertical load V of 150 psi, and the

and ϕ which are on or to the right of the line labelled $(V - L) = 100$ psi, $L = 50$ psi, and above the $(V_i - L_i)$ curve labelled 100 psi, would have the required stability and cohesion. Those paving mixtures with combinations of c and ϕ falling within the cross-hatched area of Figure 60, would be deficient in either stability or cohesion c insofar as the particular conditions of design specified for this problem are concerned.

In the Asphalt Institute diagram, Figures 57 and 60, the left-hand boundary between satisfactory and unsatisfactory paving mixtures is a vertical line. Figure 60, on the other hand, indicates that this left-hand boundary should consist of portions of two curves. Its position is

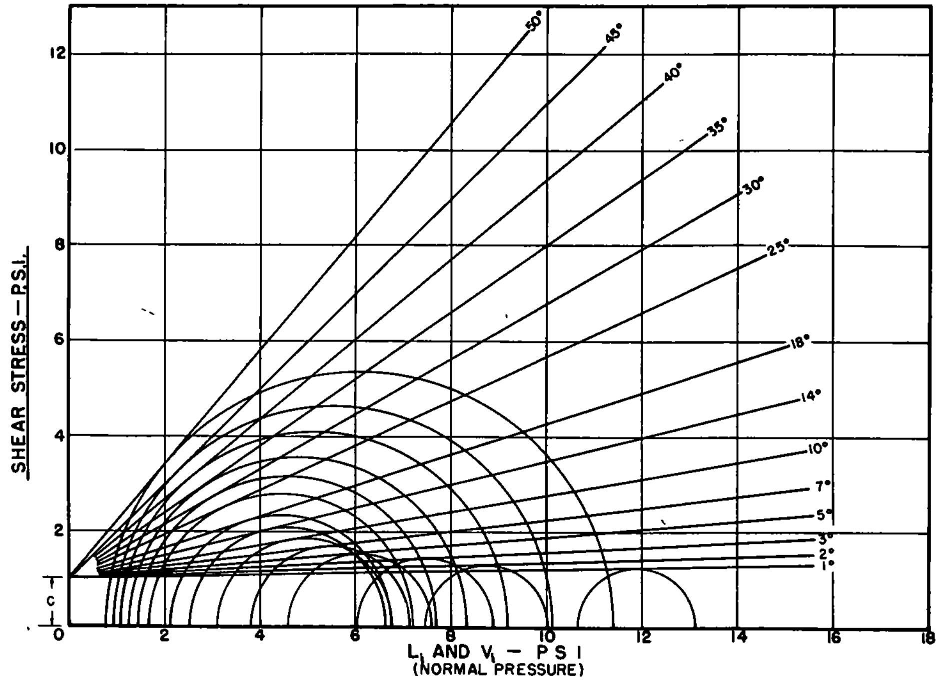

Figure 53. Mohr diagram illustrating influence of variation in angles of internal friction ϕ on values of L_l and V_l when cohesion c is constant

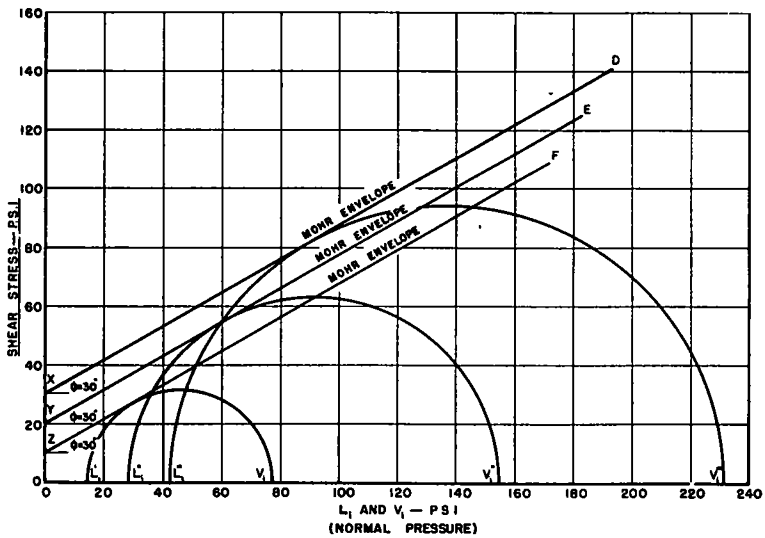

Figure 54. Mohr diagram illustrating influence of variation of cohesion c on values of L_l and V_l when angle of internal friction ϕ is constant

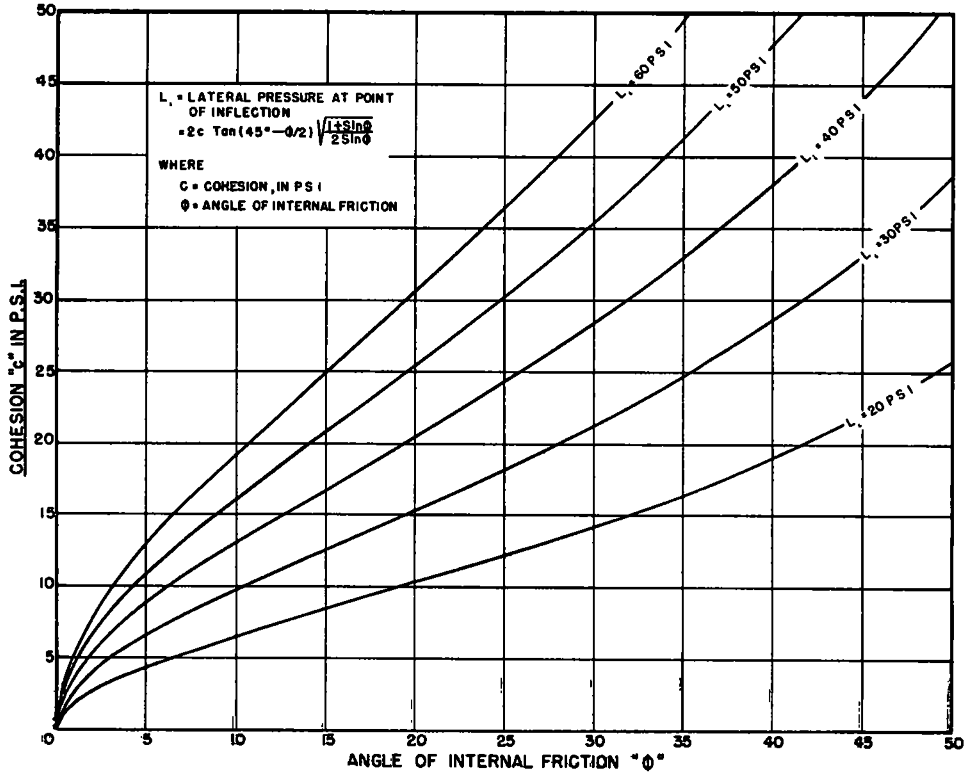

Figure 55. Relationships between cohesion c, angle of internal friction ϕ, and lateral pressure at the point of inflection L_l for triaxial compression test

not vertical, but slopes far toward the left. Consequently, Figure 60 indicates that the Asphalt Institute diagram is much too restrictive, and that satisfactory stability will be obtained for bituminous mixtures with a much wider range of c and ϕ values than it would permit.

This approach, based upon (V_l - L_l) curves, provides minimum values for cohesion c for bituminous mixture design, that appear to be in reasonable agreement with the experimental information already obtained. This is illustrated in Figure 58, where it is apparent that a (V_l - L_l) curve for about 70 psi would correspond very well with the lower boundary for satisfactory mixtures on the Asphalt Institute diagram. The lower boundary of this diagram was derived in part from empirical considerations as a result of correlating the c and ϕ values of bituminous paving mixtures determined by triaxial compression tests, with their service performance in the field, and partly from theoretical considerations based upon the mathematical theory of elasticity.

On the basis of the properties of the Mohr diagram, there is another approach to the problem of modifying the stability diagram of Figure 48 to provide the minimum values of cohesion c required for the design of bituminous paving mixtures, that might be referred to briefly. The point of inflection which occurs in each of the V - L stability curves of Figure

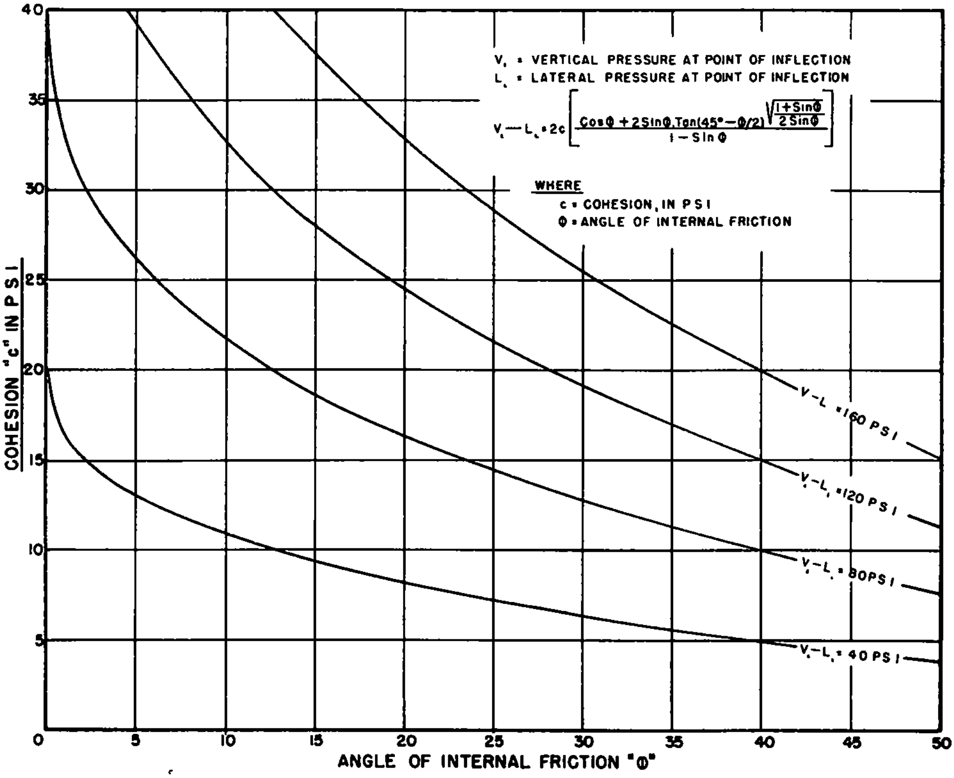

Figure 56. Relationships between cohesion c, angle of internal friction ϕ, and V_l - L_l for triaxial compression test

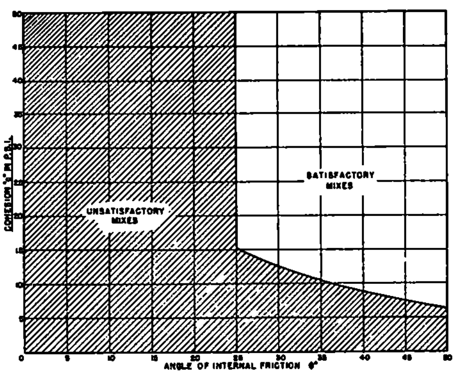

Figure 57. Design chart for asphaltic concrete based upon the triaxial compression test (The Asphalt Institute Manual on hot-mix asphaltic concrete paving)

48, might be expected to have some significance in establishing the required minimum values of cohesion c. The general equation for each of these curves is given by equation (11)

$$V - L = \frac{2\,L\,\sin\phi}{1 - \sin\phi} + 2\,c\,\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} \quad (11)$$

For all values of lateral pressure L, except $L = O$, the stability graph for any value of $V - L$ is a reverse curve. By equating the second derivative of equation (11) to zero, the value of ϕ at which the point of inflection occurs can be calculated for any set of corresponding values of $V - L$ and L. The location of these points of inflection for $V - L = 100$ psi, and for all positive values of lateral pressure L, is shown by the broken line curve of Figure 61. The general equation for the curve through the points of in-

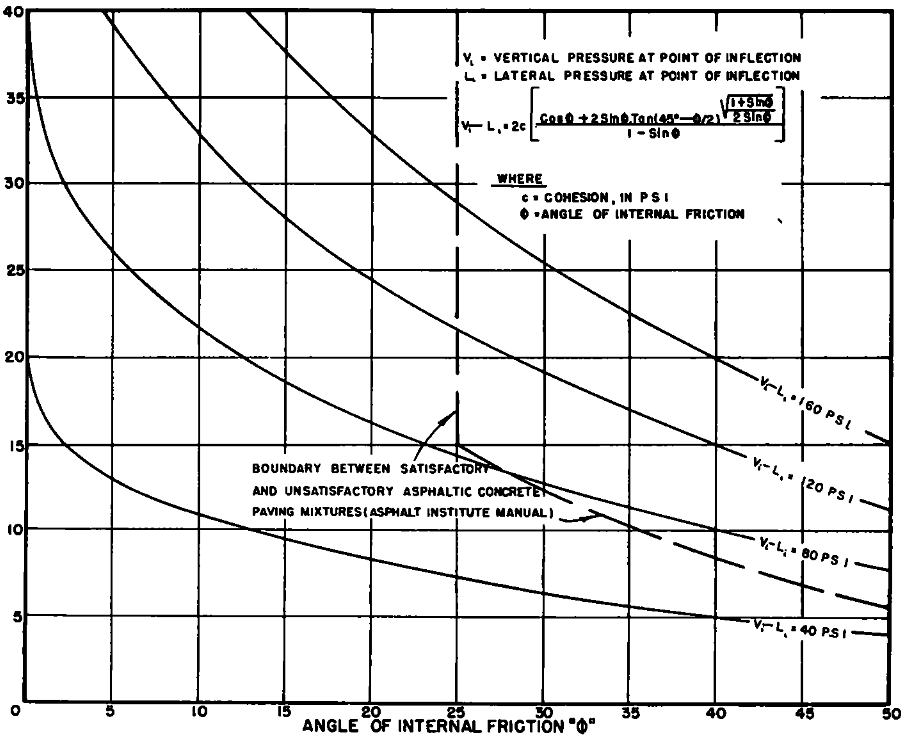

Figure 58. Graph showing the boundaries between satisfactory and unsatisfactory asphaltic concrete mixtures proposed by the Asphalt Institute, and $V_i - L_i$ values

flection for any similar stability diagram is also given in Figure 61.

The curve for $(V_i - L_i) = 100$ psi has been drawn in Figure 61. It will be observed that the broken line curve through the points of inflection on the various reverse curves for $V - L = 100$ psi in Figure 61, would require higher minimum values of cohesion c for bituminous mixtures, than would the curve for $(V_i - L_i) = 100$ psi, when ϕ is greater than 13.7 deg. That is, insofar as the range of combinations of c and ϕ values normally employed for the design of bituminous mixtures is concerned, the curve through the points of inflection would be more restrictive than the corresponding $(V_i - L_i)$ curve. This is emphasized by the cross-hatched area of Figure 61. Therefore, since the minimum values of c given by the $(V_i - L_i)$ curves appear to agree reasonably well with the required values of c determined empirically, it would seem that the $(V_i - L_i)$ curve indicated in each case, should be considered for bituminous mixture design, in preference to the corresponding curve through the points of inflection.

e. Influence of Braking Stresses - When the brakes are applied to the wheels of a moving vehicle, a horizontal thrust is developed within the pavement. A similar effect but in the opposite direction occurs when a vehicle is accelerated. This horizontal thrust decreases the effective lateral support within the pavement that is available for supporting the vertical load on the wheel. Figure

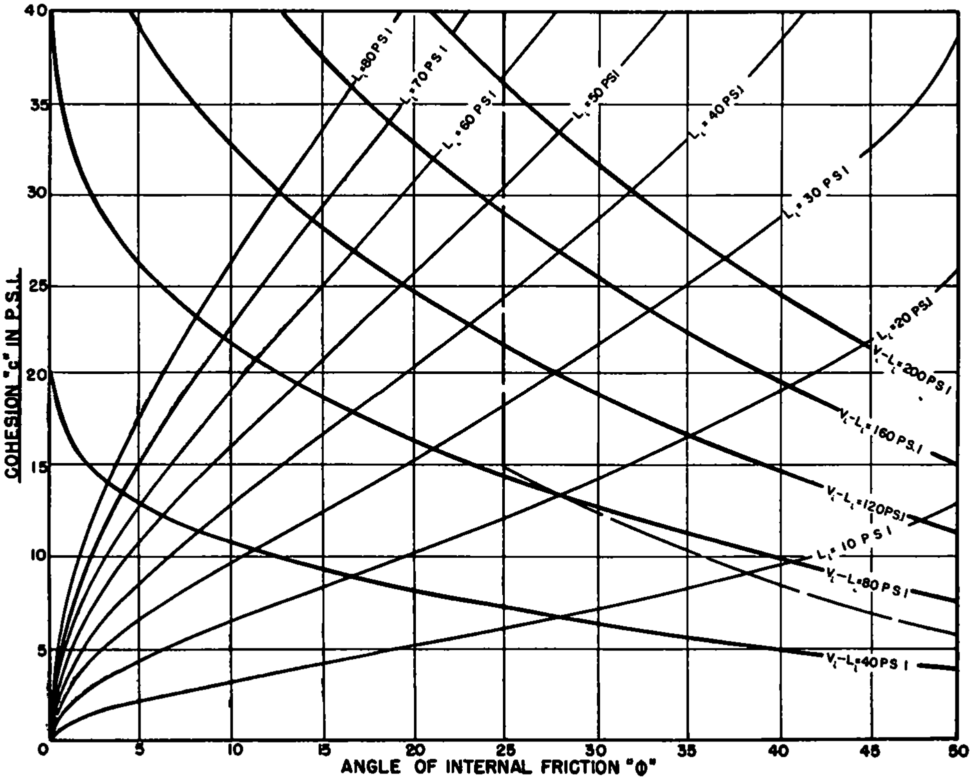

Figure 59. Chart for asphaltic concrete design based upon values of c, ϕ, L_l, and V_l - L_l derived from the triaxial compression test

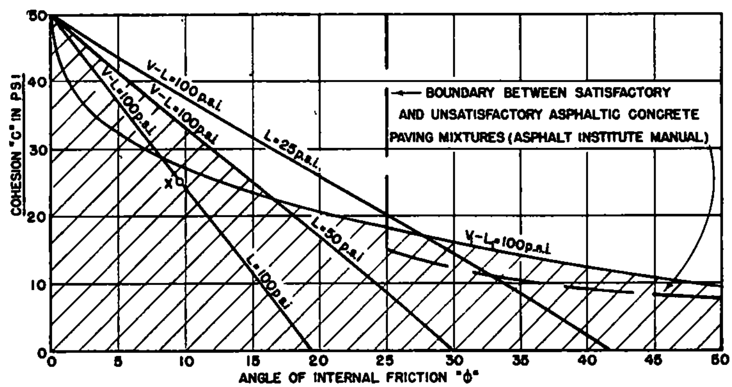

Figure 60. Use of stability curves $(V - L)$ and $(V_l - L_l)$ for flexible surface design

62 demonstrates in a quantitative manner, the influence which this horizontal thrust due to braking or accelerating stresses may have on the design of a bituminous paving mixture.

the curve $(V_t - L_t) = 125$ psi, and to the right of the curve $V - L = 125$ psi, $L = 25$ psi.

Consequently, the double hatched area represents the increase in pavement

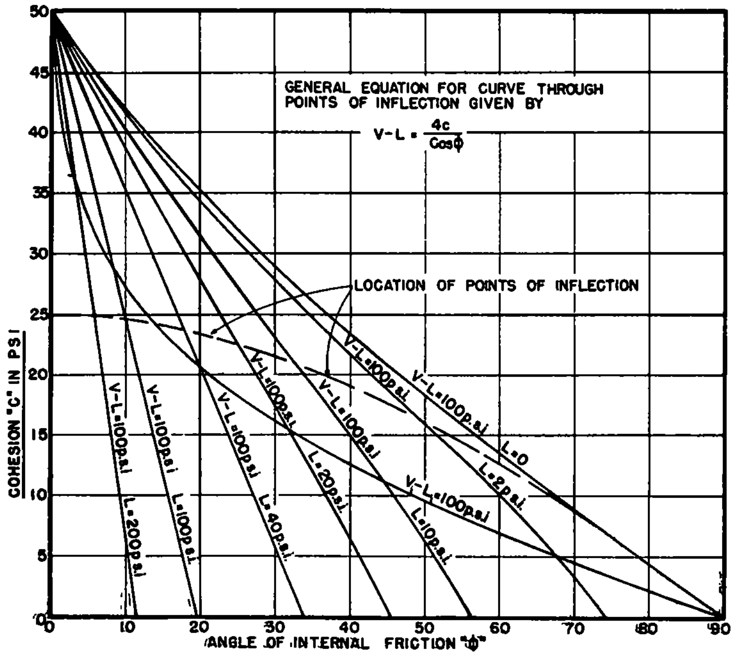

Figure 61. Location of points of inflection on all stability curves for $V - L = 100$ psi.

If the vertical load V to be carried is 150 psi, and the maximum lateral support L available under ordinary traffic conditions is 50 psi, ($V - L = 100$ psi), those paving mixtures with combinations of c and ϕ to the right of the area in single hatching in Figure 62, would have sufficient stability, that is, above the curve $(V_t - L_t) = 100$ psi and to the right of the curve $V - L = 100$ psi, $L = 50$ psi. However, if due to braking stresses, the effective lateral support L is reduced to 25 psi, then Figure 62 demonstrates that to carry a vertical load V of 150 psi, ($V - L = 125$ psi), only those bituminous mixtures with combinations of c and ϕ to the right of the double hatched area, would have the required stability, that is, above

stability that may be required because of braking or accelerating stresses (braking stresses are usually more severe). Figure 62 makes it clear why paving mixtures with little more than sufficient stability for average locations, distort badly at bus stops and traffic lights, where there is much stopping and starting of traffic.

Figure 62 demonstrates that the horizontal thrust B at the pavement surface due to braking, tends to create a shear stress H at the interface between the pavement and the base course. This shear stress H in turn, reduces the effective lateral support L within the underlying base course. Because of this lowering of effective lateral support, a more stable base course material is required to carry a given vertical load, at all points

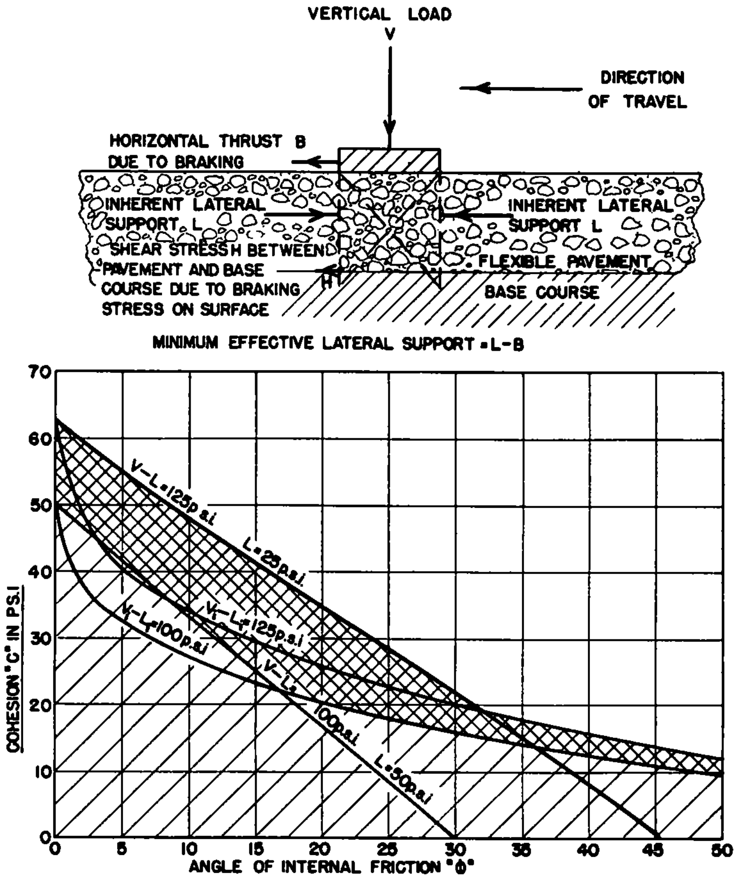

Figure 62. Influence on the design of flexible pavement mixtures of change in lateral support due to wheel load braking

where there is much stopping and starting of traffic. This can be illustrated by reference to the lower diagram of Figure 62. If the vertical load V to be carried by the lase course is 150 psi, and the maximum lateral support available under ordinary traffic conditions is 50 psi, ($V - L = 100$ psi), only those base course materials, with combinations of c and ϕ to the right of the curve labelled $V - L =$ 100 psi, and $L = 50$ psi, would have the required stability. However, if due to braking stresses at the pavement surface, the effective lateral support in the base course is reduced to 25 psi, Figure 62 demonstrates that to carry a vertical load V of 150 psi, ($V - L = 125$ psi), only those base course materials with combinations of c and ϕ to the right of the line designated $V - L = 125$ psi, $L = 25$ psi, would have the necessary stability.

Consequently, at bus stops, traffic lights, and all other locations where there is much braking or accelerating of traffic, not only must the bituminous pavement have greater stability to withstand the stresses of stopping and starting, but the underlying base course material must have greater stability than would otherwise be necessary. For similar reasons, this is also true of sections of pavement on slopes, particularly with steep gradients, as compared with level areas, and for the pavement around curves where considerable side thrust may be exerted by high speed vehicles.

To make the presentation as simple as possible, the above discussion has avoided consideration of secondary vertical reactions which are caused by braking stresses. However, these could be added to the other vertical forces involved, and the same method of solution followed.

GENERAL

1. It should be particularly noted that the development based upon the triaxial compression test which has been outlined here, places the design of base courses and flexible wearing surfaces on a pounds per square inch basis.

2. It is to be observed that Figure 60 can be employed by itself to determine the minimum value of c for a purely cohesive material, the minimum value of ϕ for a purely granular material, or the required combinations of c and ϕ needed by materials having both cohesive and granular properties to function as either base or wearing courses. One extremity of the curve $(V - L) = 100$ psi, $L = 50$ psi, in Figure 60, cuts the c axis at $c = 50$ psi, the minimum value of cohesion which a purely cohesive material must have for this particular problem. The other extremity of this $(V - L)$ line cuts the ϕ axis at $\phi = 30$ deg., the minimum value of ϕ required for a purely granular material. Only the combinations of c and ϕ to the right of this $(V - L)$ line satisfy the stability requirements of this problem for base course materials, while the curve for $(V_1 - L_1) = 100$ psi, imposes the special restrictions which are needed to provide suitable paving mixtures for the surface course. Consequently, the special stability diagrams of Figures 43 and 46, for purely cohesive and purely granular materials respectively, are unnecessary, since the same information can be derived from the general stability diagrams of Figures 48, 50 and 60 etc.

3. The amount of lateral support L which can be developed by a base course or flexible pavement for an airport or highway is largely unknown at the present time. However, by means of the development which has just been outlined, it would appear that the lateral support L for existing base courses and bituminous pavements can be evaluated by suitable investigations which include both field observations and laboratory tests. Values of c and ϕ can be measured by the triaxial test performed on representative samples from projects for which the base course layer, or surface layer, or both, have shown definite instability, from projects showing evidence of incipient instability, and for those that are stable. The maximum unit vertical load V supported on each project can be measured or calculated from vehicle tire pressure, corrected by a suitable factor for dynamic loading where necessary *(13)*. This would leave the lateral support L as the only unknown variable in the general stability equation (11), and its value could therefore be calculated.

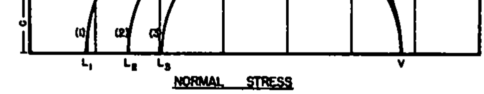

Figure 63. Mohr circles representing unstable, equilibrium, and stable combinations of V and L values for a given material under stress

The value of the available lateral support L could also be determined graphically by means of the Mohr diagram, Figure 63. The Mohr envelope is drawn from the values of c and ϕ measured for the sample by the triaxial test, and the value of the vertical load V to be carried is marked on the abscissa. A Mohr circle (2) passing through V and just touching the Mohr envelope is drawn by trial and error, or by calculating the corresponding value of L from equation (11). Mohr circles (1) and (3) through V, cutting the Mohr envelope and within it, respectively, are also shown in Figure 63. Mohr circle (1) represents a condition that would result in failure of the material in service under vertical load V, because sufficient lateral support L_1, could not be developed for stability. Mohr circle (2) indicates a condition where there is just sufficient lateral support L_2, to avoid failure. It represents a condition of equilibrium. Mohr circle (3) indicates a highly stable condition since more lateral support, L_3, is available than the minimum required for equilibrium. Consequently, if the sample of material came from a failed area, it would be represented by Mohr circle (1), if from an area of incipient failure, by Mohr circle (2), and from a highly stable area, by Mohr circle (3). From information of this nature from many projects, reasonable estimates of the amount of lateral support L available under various conditions in the field could eventually be made.

It is quite probable that subgrade, base course, and bituminous surfacing materials develop a definite structure under traffic and with the passage of time, that adds to their strength, and which is broken down if samples of these materials are remolded for the triaxial test. Consequently, the c and ϕ values for the material in place may be different than those measured for remolded samples. Since it is the values of c and ϕ for the material in place that are required, if reasonably accurate values of available lateral support L are to be determined, the triaxial test should be made on either undisturbed base course and pavement samples, or on remolded samples that have been compacted in such manner that the structure of the material in place is duplicated. A time factor may also have to be considered when attempting to duplicate in laboratory test specimens, the structure developed in any given material under traffic in the field.

Dynamic factors associated with moving vehicles, and other variables, may make the problem more complicated, but the methods just described for evaluating the degree of lateral support L available under various conditions, appears to be reasonable as a first approach. The amount of lateral support L that can be mobilized by the material in any given layer may depend upon its composition, density, liquid content, temperature, etc., the thickness of the layer in question, the thickness, composition, etc., of the overlying layers, the nature of the underlying material, size of contact area of the applied load, etc. However, if the problem were carefully studied, it might eventually be possible to prepare suitable tables of values for available lateral support L, in which all of these different variables are taken into account.

4. While very little information can be found concerning the amount of lateral support L available within subgrade, base course, or surfacing materials under stress, nevertheless, insofar as the design of bituminous paving mixtures are concerned, there are at least two existing indications of the amount of lateral support L that might be safely assumed for design.

Figure 57, taken from the Asphalt Institute Manual on Hot Mix Asphaltic Concrete Paving, indicates the combinations of c and ϕ that asphaltic concrete must have for satisfactory stability, and those combinations that are unstable. This diagram has been checked on the basis of the performance in the field of asphaltic concrete pavements for which corresponding c and ϕ values have been measured in the laboratory, and is considered to be satisfactory *(12)*. It will be observed that the point of intersection of the lower with the vertical boundary in this diagram occurs at c = 15 psi, ϕ = 25 deg. From Figure 59, it would appear that paving mixtures to the left of this intersection, even though on or above the projection to the left of the lower boundary, are unsatisfactory because the amount of lateral support L which they would require for stability, cannot be developed in the field. Figure 59 indicates that the curve for L_i = 25 psi would pass through this point of intersection (c = 15 psi, ϕ = 25 deg.). Consequently, in the absence of more definite information at this time, it would seem reasonable to assume on the basis of Figures 57 and 59, that the design of hot-mix asphaltic concrete paving mixtures could be based upon the assumption that the maximum amount of lateral support L available is 25 psi.

The second method for arriving at a reasonable value of lateral support L for bituminous mixture design is based upon the fact that for any given mixture, the value of L that can be developed in the field is always greater than $2c$, that is, greater than twice the cohesion c of the paving mixture, except for occasional poorly designed mixtures for which L might approach the value of $2c$.

That the amount of lateral support L which a bituminous pavement can develop in the field is normally greater than twice the cohesion c of the paving mixture, can be quite easily illustrated. Figure 64(a) represents the principal stresses and shear stresses that are developed in a bituminous pavement under load, when the weight of the material is neglected. The stresses acting on element (2) indicate that the lateral pressure L exerted by element (1) on element (2), is resisted by the shear stress s_c acting along the diagonal plane of element (2). Figure 64 (b) and (c) illustrate the principal and shear stresses acting upon elements (1) and (2), respectively. Figure 64 (d) is a Mohr diagram representing the stresses acting on element (2), for a bituminous paving mixture having values of c and ϕ that result in the Mohr envelope indicated. L is the major principal stress acting on element (2), and the minor principal stress is zero, if the weight of the element and other factors are neglected. The Mohr circle of rupture representing these combinations of stress under equilibrium conditions, has the radius $L/2$. It is apparent from the geometry and trigonometry of the Mohr diagram of Figure 64(d), that the radius of the Mohr circle of rupture, $L/2$, is greater than the shear stress, s_c, on the critical plane. It is also obvious that s_c is greater than the cohesion c. Therefore

$$L/2 > s_c > c \tag{17}$$

from which it follows that

$$L > 2\,s_c > 2c \tag{18}$$

Consequently, the amount of lateral support L that a bituminous pavement can develop in the field, is greater than twice the cohesion c of the bituminous paving mixture.

Even for the worst condition that could develop in a pavement in the field, that is, when the angle of internal friction ϕ is or becomes zero, Figure 64(c) demonstrates that the maximum lateral support L available cannot be less than twice the cohesion c.

Figure 64(d) and equation (18) indicate that by assuming that $L = 2c$ for design purposes, very conservative values of lateral support L will normally be employed. Furthermore, it should be noted that the weight of element (2) has been neglected in this development. If this weight were taken into consideration, the assumption for purposes of design that $L = 2c$ becomes still more conservative.

The following example of the design of a bituminous mixture on the basis that the lateral support L available in the field is not less than twice the cohesion c of the paving mixture, is provided. Suppose that when a trial or proposed paving mixture is tested in triaxial compression, and the Mohr envelope is plotted, that the value of cohesion c is found to be 10 psi. For design purposes, it is assumed that the maximum lateral support L that can be developed = $2c$ = 20 psi. If the vertical load V to be carried by the pavement is 80 psi, then the re-

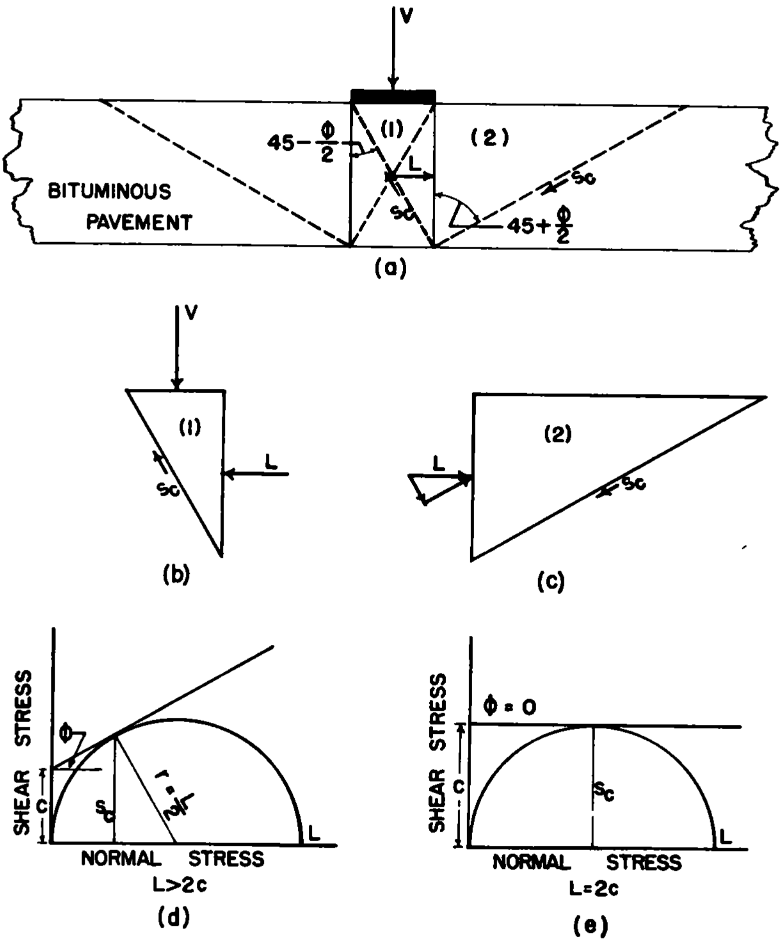

Figure 64. Illustrating that the available lateral support L for a bituminous pavement is generally greater than twice the cohesion c of the paving mixture

quired stability $V - L = 60$ psi. In Figure 65, the stability curve for $V - L = 60$ psi and $L = 20$ psi has been drawn, together with the $(V_l - L_l)$ curve for 60 psi which indicates the minimum cohesion necessary for these particular conditions of design. In addition, the value of the cohesion $c = 10$ psi is shown in Figure 65 as a broken line parallel to the abscissa. Figure 65 indicates that to meet the conditions of this particular design problem, a paving mixture with cohesion $c = 10$ psi must at the same time have an angle of internal friction $\phi = 28$ deg. or higher. It can be seen that when $c = 10$ psi, an angle $\phi = 28$ deg. is the smallest that will provide a paving mixture with coordinates of c and ϕ that lie on or to the right of the stability curve for $V - L = 60$ psi, $L = 20$ psi, and is at the same time on or above the curve for $(V_l - L_l) = 60$ psi.

5. It should be emphasized that the values of c and ϕ obtained for any given material depend upon the procedure employed for the triaxial test. This fact

has been carefully pointed out by Endersby *(14)* and others. No standardized procedure for this test has yet been established, and several different methods are being employed. The size of sample and its dimensions, the method of preparation of the sample, the speed of testing, the size of the largest particle, absence or freedom of drainage, the temperature of test, and the procedure for applying lateral and vertical pressures, are some of the variables that must be considered. For the design of base courses and bituminous mixtures for stability, for example, the procedure devised for the triaxial test would seem to require close correlation with the conditions of loading to which the materials are exposed on a roadway or runway.

results obtained by investigators in different laboratories can be placed on a common basis of comparison.

6. Figure 66 illustrates how an extrusion test, or any of the ordinary compression tests, could register high stability for a sample of a bituminous paving mixture in the laboratory, which would later be found to be unstable in the field. If a bituminous pavement for a given project can develop a maximum lateral support L of 50 psi, and must carry a vertical load of 150 psi, only those paving mixtures with combinations of c and ϕ to the right and above the cross-hatched area of diagram A would have the required stability and cohesion.

An extrusion test or any one of the ordinary compression tests might register

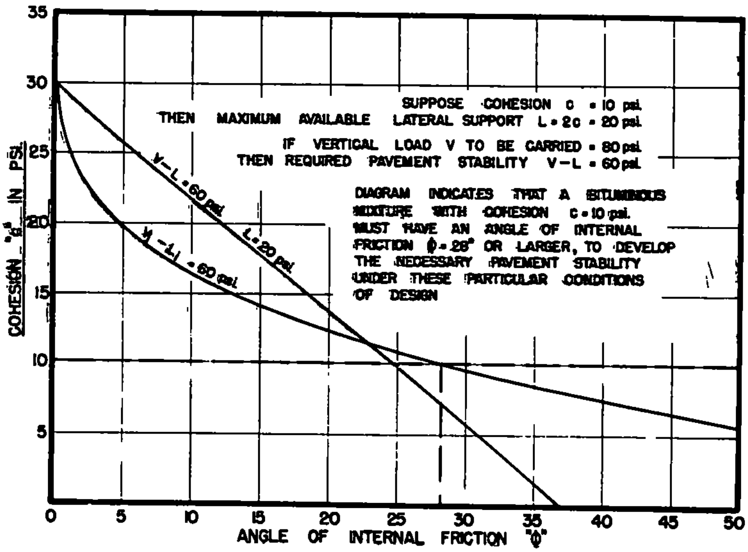

Figure 65. An example of bituminous mixture design by the triaxial method

Consequently, before the stability equations and diagrams based on a straight line Mohr envelope, which have been outlined above, can be employed, a procedure for the triaxial compression test must be devised that will provide values for c and ϕ which are truly representative of conditions as they exist in the field. A standardized procedure for the triaxial test must also be developed before the

high stability for a bituminous mixture having the c and ϕ values, c = 25 psi and ϕ = 9 deg. 45 min., represented by point X in diagram A of Figure 66. For point X, diagram A indicates a $V - L$ value of 100 psi, if a lateral support L of 100 psi can be developed. That is, a vertical load V of 200 psi could be supported by a bituminous mixture represented by point X, if it could develop a lateral

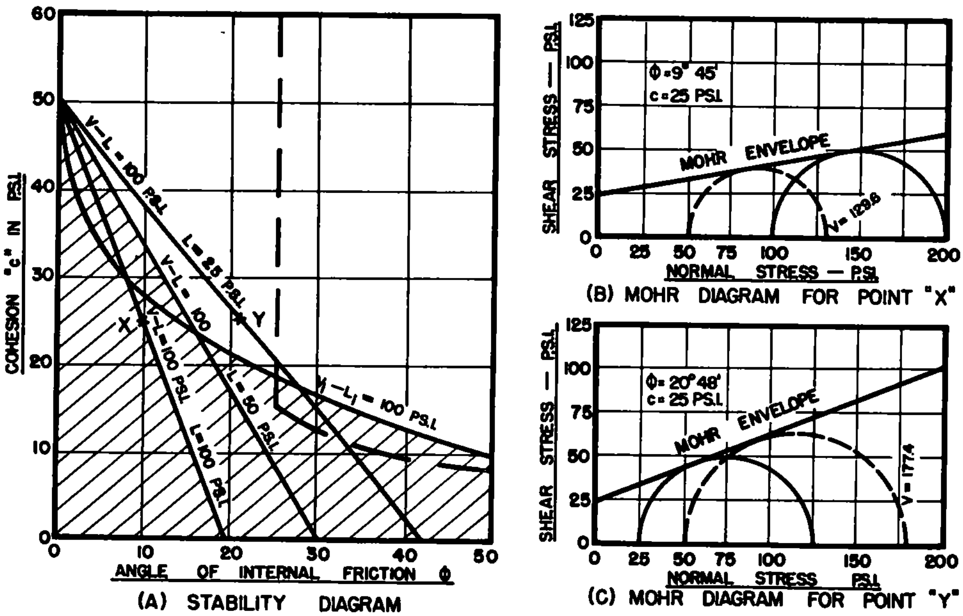

Figure 66. Conditions of stability and instability in terms of c and ϕ which govern the selection of base course and flexible surfacing materials with respect to a given requirement

support of 100 psi. This combination of V and L values is represented by the full line Mohr circle in diagram B of Figure 66. However, the maximum lateral support L available, is only 50 psi. The broken line Mohr circle of diagram B of Figure 66 indicates that the bituminous mixture represented by point X could support a vertical load V of only 129.6 psi, if the lateral support were 50 psi. According to the conditions of the problem, it must be capable of supporting a vertical load V of 150 psi at a lateral support of 50 psi. Consequently, the bituminous mixture represented by point X in diagram A of Figure 66 does not have the stability required for the conditions of this project.

In the extrusion test, the sample is rigidly confined within a steel cylinder when vertical load is applied. The amount of lateral support provided for the sample is therefore indeterminate, probably variable from mixture to mixture, and likely quite high. For any of the ordinary compression tests, the lateral support provided is zero, or essentially so. Consequently, since neither the extrusion nor the ordinary compression tests provide test conditions similar to those to which a bituminous pavement is subjected in the field, they may give entirely erroneous measurements of the stability which a bituminous mixture will be able to develop under service conditions, as the above example has illustrated.

Section C of Figure 66, is a Mohr diagram for point Y in part A of this figure. Point Y represents corresponding c and ϕ values of c = 25 psi, and ϕ = 20 deg. 48 min. Point Y indicates that if the lateral support L available is 25 psi, the maximum vertical load V which can be carried is 125 psi. This is illustrated by the full line Mohr circle in diagram C. The question might be asked, that if the bituminous mixture with the c and ϕ values represented by point Y in diagram A can support a vertical load of only 125 psi when the lateral support is 25 psi, will it be able to carry a vertical load of 150 psi, as required by this problem, when the lateral support is 50 psi? The answer is given by the broken line Mohr circle of diagram C, which shows that for a lateral support of 50 psi, this bituminous mixture will be stable under a maximum vertical load of 177.4 psi.

7. While stability has been considered in terms of $(V - L)$ values in this paper, it is to be noted that each $(V - L)$ value can be converted into shearing resistance. The maximum shearing resistance that a material can develop for corresponding values of V and L is $(V - L)/2$, and it occurs on planes making an angle of 45 deg. with the direction of the principal stresses. However, the maximum shearing resistance that can be developed on the actual plane of failure, sometimes called the critical plane, is given by the equation,

$$s_c = \frac{V - L}{2} \cos \phi \tag{19}$$

where s_c, is the maximum shearing resistance on the critical plane, and the other symbols have the significance already attributed to them.

If required, general stability equation (11) can be very easily expressed in terms of s_c rather than $(V - L)$ values, when it becomes

$$s_c = L \sin \phi \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} + c(1 + \sin \phi) \tag{20}$$

The diagram corresponding to equation (20) is illustrated in Figure 67.

8. Equation (9) for the stability of purely granular materials, can be rearranged into the following form

$$\frac{V}{L} = \frac{1 + \sin \phi}{1 - \sin \phi} \tag{21}$$

or as

$$V = L\left(\frac{1 + \sin \phi}{1 - \sin \phi}\right) \tag{22}$$

Equation (21) indicates that the value of the ratio V/L for purely granular materials is a function of the angle of internal friction ϕ, while equation (22) shows that the maximum vertical load V that can be carried by a mass of purely granular material depends upon the lateral support L and the angle of internal friction ϕ.

When equation (21) is plotted in terms of V/L versus angle of internal friction ϕ, the stability diagram of Figure 68 is obtained.

If a base course is to carry a vertical load V of 150 psi, and can develop a lateral support L of 50 psi, the value of the ratio V/L is 3, and Figure 68 indicates that a purely granular material with an angle of internal friction ϕ = 30 deg. or greater, would be required to provide the necessary stability. The same solution to this problem was given by Figure 46.

Consequently, the mathematical equation of stability for purely granular materials can have the form of either equation (9) or equation (21). Furthermore, the stability diagram for purely granular materials is represented by either Figure 46 or Figure 68.

9. From the geometrical and trigonometrical properties of the general Mohr diagram, illustrated in Figures 47 and 51, other equations than those already developed, can be derived to express relationships between various combinations of the quantities vertical load V, lateral support L, maximum shear stress on the critical plane (the plane of failure) s_c, maximum normal stress on the critical plane n_c, cohesion c, and angle of internal friction ϕ, together with the V_i, L_i, n_i, and s_i quantities that can be derived from the point of inflection in Figure 51.

Of the relationships not already discussed, that between V, L, c, and ϕ obtained by rearranging equation (11) is of particular interest.

$$V = L\left(\frac{1 + \sin \phi}{1 - \sin \phi}\right) + 2c \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \tag{23}$$

According to equation (23), if the ver-

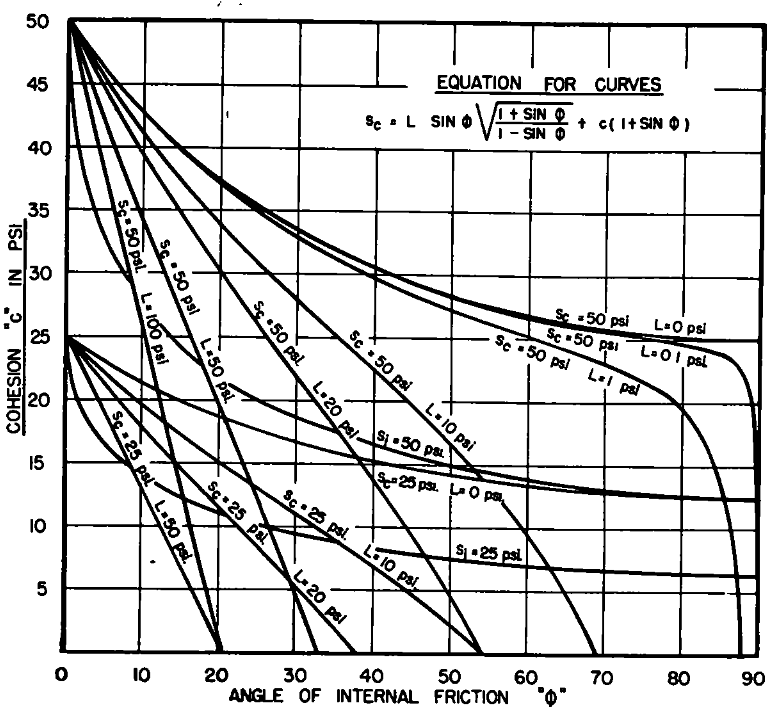

Figure 67. Relationship between c, ϕ, L and s_c for materials having positive values for c and ϕ in triaxial compression test

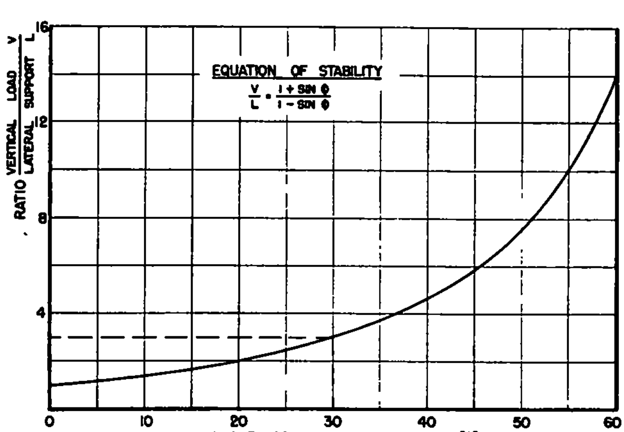

Figure 68. Stability diagram in terms of ϕ and $\frac{V}{L}$ for materials having zero cohesion in triaxial compression test

tical load V to be carried is specified, and the amount of lateral support L is known, the combinations of c and ϕ which the materials under stress must possess to avoid failure of the material, can be calculated. Similarily, if a certain vertical load V is to be carried, and the values for c and ϕ for the particular material under stress have been measured, the minimum value of lateral support L required to prevent failure of the structure can be calculated.

A diagram based upon equation (23) is illustrated in Figure 69. The curve for V = 100 psi and L = 20 psi, for example, indicates that only those materials with combinations of c and ϕ which lie on and to the right of this curve would have the necessary stability under these particular conditions of vertical stress V and lateral support L.

The general equation expressing the interrelationship between the normal pressure on the critical plane, n_c, L, c and ϕ is

$$n_c = L\ (1 + \sin \phi) + c \cos \phi \qquad (24)$$

and that expressing the interrelationship between s_c, n_c, c, and ϕ is the well known Coulomb equation.

$$s_c = c + n_c \tan \phi \qquad (25)$$

The equation for n_l and s_l, the values of n_c and s_c respectively at the point of inflection, Figure 51, are

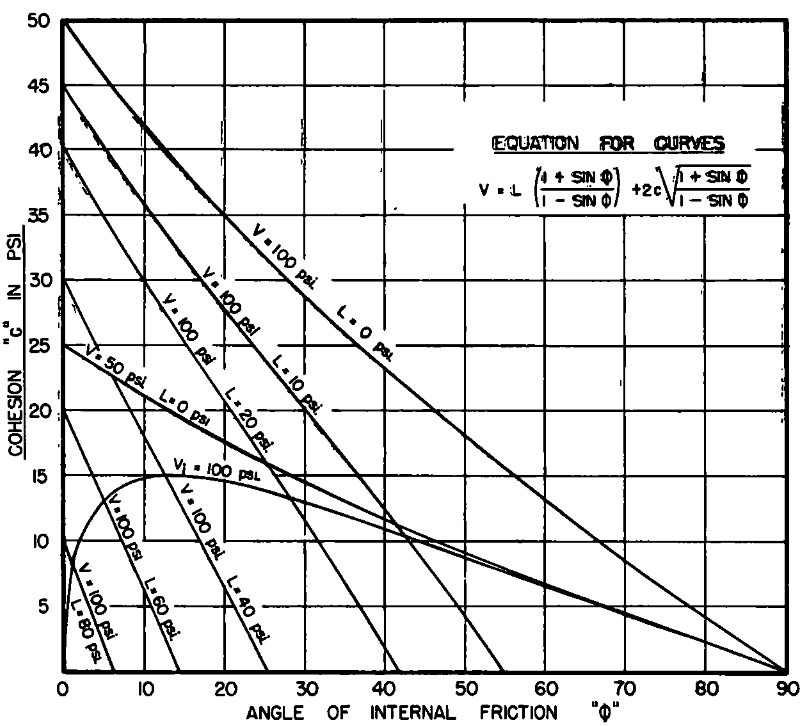

Figure 69. Relationship between c, ϕ, L and V for materials having positive values for c and ϕ in triaxial compression test

$$n_l = c \cos \phi\ (2 \sqrt{\frac{1 + \sin \phi}{2 \sin \phi}} + 1) \qquad (26)$$

$$s_l = c(1 + \sin \phi + \sqrt{2 \sin \phi}\sqrt{1 + \sin \phi}) \qquad (27)$$

Diagrams which indicate graphically the relationships expressed in equations (24) and (25), are illustrated in Figures 70 and 71, respectively, and the corresponding n_l and s_l curves are also shown.

are kept constant in equation (28), and n_c and V are kept constant in equation (29), it should be noted that the value of the lateral support L is changing whenever c or ϕ or both vary.

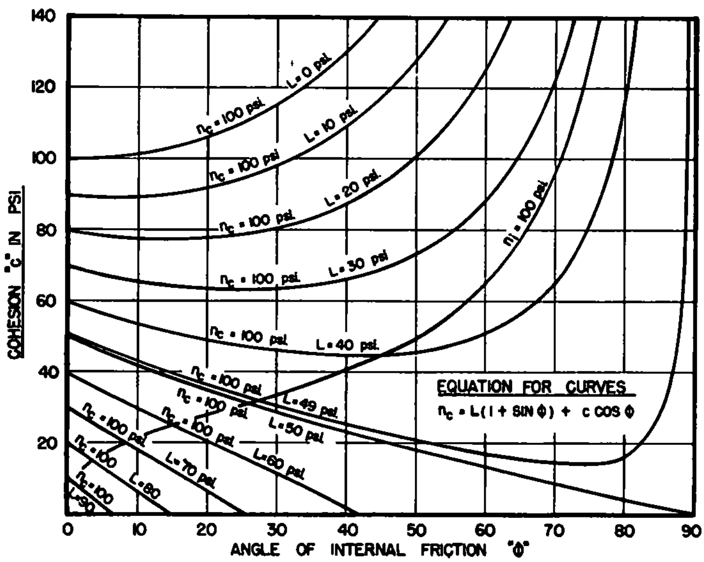

Figure 70. Relationship between c, ϕ, L and n_c for materials having positive values for c and ϕ in triaxial compression test

Equations (20) and (24) can be written in terms of vertical load V rather than lateral support L, as

$$s_c = V \sin \phi \sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}} + c\ (1 - \sin \phi) \qquad (28)$$

and

$$n_c = V\ (1 - \sin \phi) - c \cos \phi \qquad (29)$$

Graphs demonstrating the relationships of both equations (28) and (29) in rearranged form, are illustrated in Figures 72 and 73 respectively.

It should be observed in connection with equations (28) and (29) and Figures 72 and 73, that negative values of L are encountered for certain combinations of values for the variables in these equations. Furthermore, whenever s_c and V

Experimental data on materials tested in a shear box are obtained in terms of s_c, n_c, c, and ϕ, while data on the same materials tested in triaxial shear are obtained directly in terms of V, L, c, and ϕ. Consequently, it might be desirable to rearrange the various equations containing these different variables depending upon whether the source of the data is from the shear box or from the triaxial test. That is for example, equation (29) might be preferred in its present form for data obtained from the shear box, but rearranged into the form

$$V = \frac{n_c}{1 - \sin \phi} + \frac{c \cos \phi}{1 - \sin \phi}$$

$$= \frac{n_c}{1 - \sin \phi} + c \frac{1 + \sin \phi}{1 - \sin \phi} \qquad (30)$$

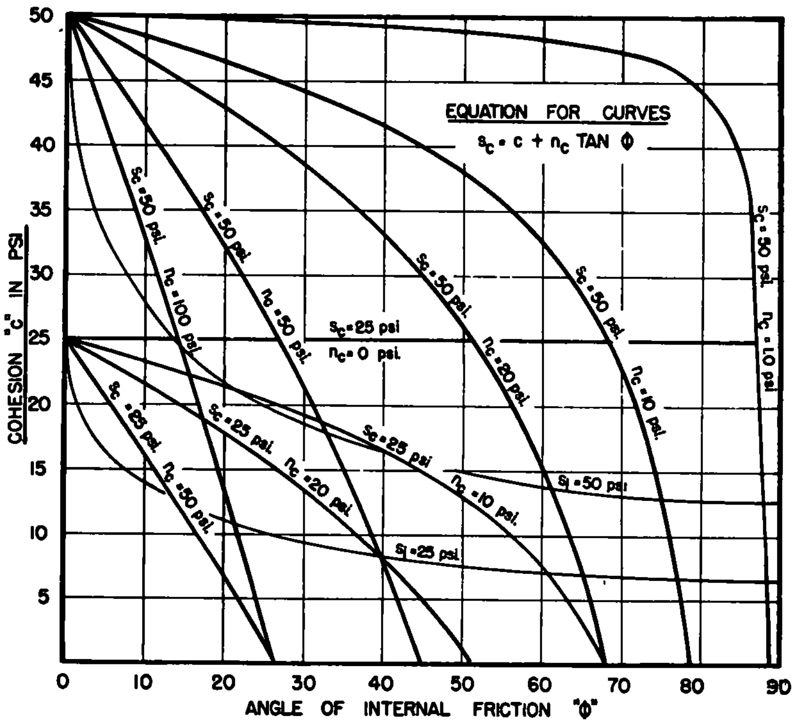

Figure 71. Relationship between c, ϕ, n_c and s_c for materials having positive values for c and ϕ in triaxial compression test

for data obtained directly from the triaxial shear test.

10. Figure 59 is a diagram of $(V_l - L_l)$ and L_l curves. In Figure 60 the curve for $(V_l - L_i)$ = 100 psi cuts across the lines for $V - L$ = 100 psi and L = 25, 50 and 100 psi. The intersection of the curve $(V_l - L_l)$ = 100 psi with the line $V - L$ = 100 psi, L = 50 psi, of Figure 60, is also the point of intersection of $(V_l - L_l)$ = 100 psi, L_l = 50 psi in Figure 59. That is, the point of intersection of the curves for $(V_l - L_i)$ = 100 psi and L_l = 50 psi in Figure 59, must lie on the stability curve for $V - L$ = 100 psi, L = 50 psi of Figure 60. Similarly, the point of intersection of the curves for $(V_l - L_l)$ = 100 psi and L_i = 25 psi of Figure 59, must lie on the stability curve for $V - L$ = 100 psi, L = 25 psi of Figure 60, and so on.

While the stability curves, $V - L$ = 100 psi, L = 25 psi; $V - L$ = 100 psi, L = 50 psi; and $V - L$ = 100 psi, L = 100 psi, etc. are actually reverse curves, they are very nearly straight lines for the intermediate and higher values of L. From Figure 48 some conception of the very large number of these stability curves required to represent various combinations of vertical load and lateral support may be obtained. The calculation of the exact location of these stability curves requires some time for each problem, and it would be worth while to have a rapid method for locating their position for all magnitudes of vertical load V, and lateral support L.

It can be shown that a diagram of $(V_l - L_i)$ and L_l curves forms a general stability diagram on which the stability curve for any combination of values for vertical load V and lateral support L can be very quickly located. This will be illustrated by means of Figure 74.

The intersection of the curve for $(V_l - L_l)$ = 80 psi with the curve for L_l = 60 psi must also be a point on the

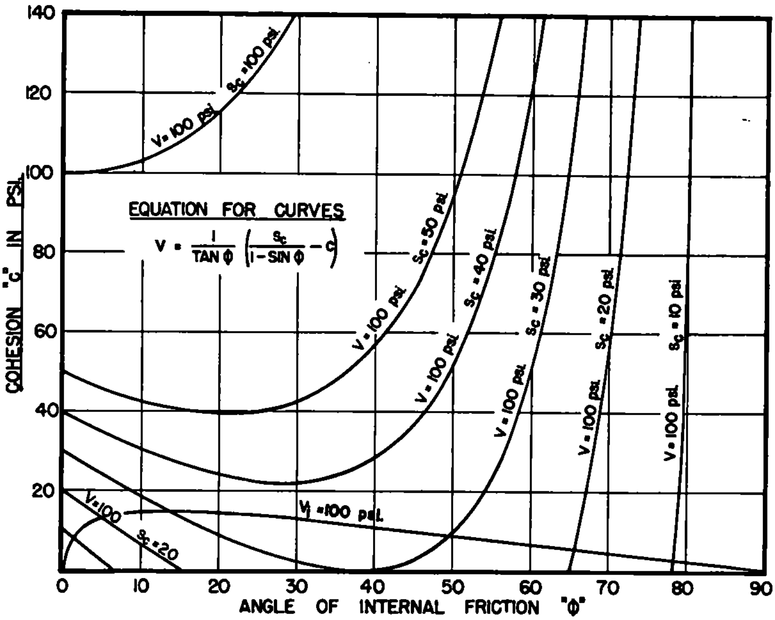

Figure 72. Relationship between c, ϕ, s_c and V for materials having positive values for c and ϕ in triaxial compression test

It has been already stated that for all except the small values of L, while the stability curves in terms of $V - L$ and L, e.g. Figure 48, are actually reverse curves, they are very nearly straight lines. The direction of a straight line is fixed, if the location of any two points on the line is known.

Suppose that the location for the stability curve for V = 140 psi and L = 60 psi is required, Figure 74. Then $V - L$ = 80 psi. The stability curve required therefore is for $V - L$ = 80 psi, L = 60 psi. The intersection of the required stability curve with the c axis is easily calculated from equation (6)

$$V - L = 2c \tag{6}$$

That is, one extremity of the required stability curve is located on the c axis at c = 40 psi.

required stability curve for the reasons given earlier in this section. Consequently, a straight line drawn from c = 40 psi through the intersection of $(V_l - L_l)$ = 80 psi and L_l = 60 psi, Figure 74, represents the location of the required stability curve for $V - L$ = 80 psi, L = 60 psi within the range of accuracy required for most practical problems. Similarly, the straight line drawn from c = 40 psi through the point of intersection of the two curves $(V_l - L_l)$ = 80 psi, L_l = 40 psi represents very nearly the location of the required stability curve for $V - L$ = 80 psi, L = 40 psi, that is the stability curve for V = 120 psi and L = 40 psi.

Therefore, the network of $(V_l - L_l)$ and (L_l) curves similar to that of the diagrams of Figures 59 and 74, provides a simple and rapid method for determining the location of the stability curves for

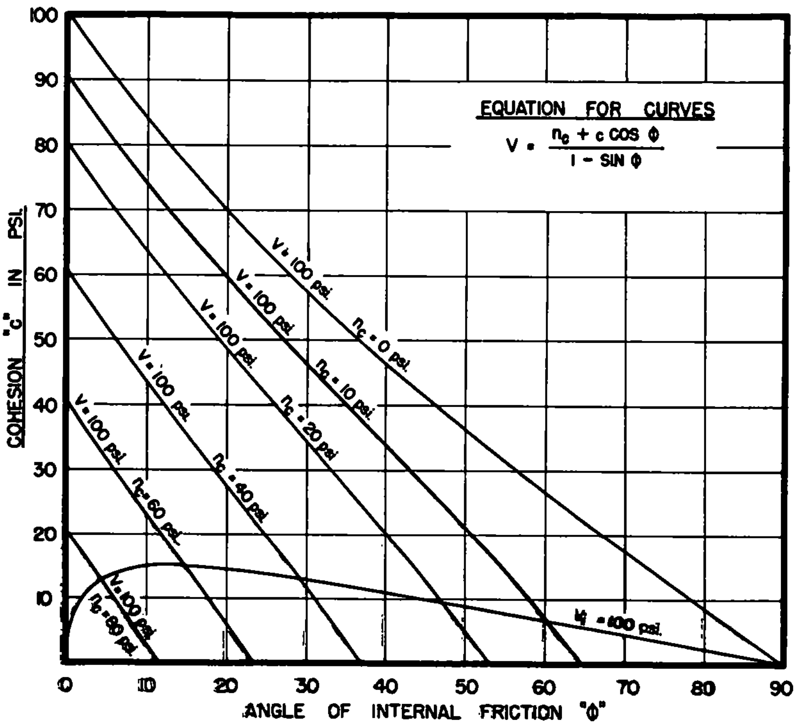

Figure 73. Relationship between c, ϕ, n_c and V for materials having positive values for c and ϕ in triaxial compression test

all combinations of values for V - L and L.

Similarly, the network of s_l and L_l curves provides a rapid method for locating the curves for all combinations of s_c and L values from equation (20), as shown by Figure 75, although in this case the range of accuracy is less for the smaller values of L, due to the greater curvature of the reverse curves for corresponding s_c and L values in this region.

11. This same procedure based upon the principal stress, shear stress, normal stress, c and ϕ values for the point of inflection, can be of similar assistance for locating the curves for other equations previously developed.

A network of V_l and L_l lines can be employed for locating the curves for all combinations of V and L values from equation (23).

n_l and L_l lines can be used for rapidly locating the curves for combinations of n_c and L values from equation (24), except when n_c is greater than $2L$.

The network of s_l and n_l lines can be utilized to locate the curves for combinations of s_c and n_c values from equation (25), but only when s_c is less than about $2n_c$.

The V_l and s_l lines are entirely unsatisfactory for locating the curve for any combination of V and s_c values from equation (28).

The network of V_i and n_i lines can be employed to locate the curves for all combinations of V and n_c values from equation (30).

12. It should be noted that the development presented in this section is concerned with evaluating the stability of various materials. In the design of bituminous mixtures, other characheristics

such as density, durability, etc., must always receive a great deal of attention. However, after all these other matters have been given due consideration, the development which has been outlined here makes it possible to determine whether or not the resulting paving mixture will have the stability required, and if not, in what direction its design must be modified in order that it will have adequate stability.

every other respect. The triaxial method of design will probably indicate that a great many aggregrate materials have been needlessly rejected in the past. This method will also indicate precisely in which direction an inferior bituminous paving mixture must be improved in order that it will develop the necessary minimum stability.

13. The development presented in this section would seem to have value for

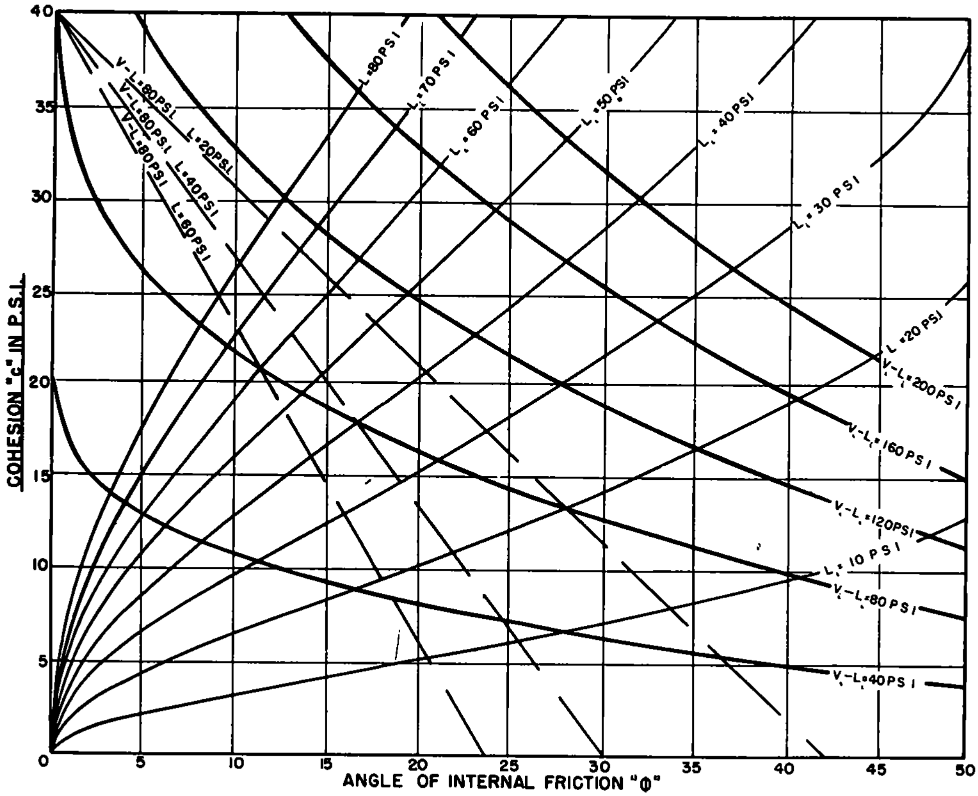

Figure 74. Stability diagram based upon values of V_l - L_l, L_l, V - L, L, c and ϕ derived from the triaxial compression test

As a result of the rapid depletion of readily accessible deposits of good gravels, materials previously considered inferior must now be investigated as suitable aggregrates for bituminous paving mixtures. The triaxial method outlined in this section, will indicate whether paving mixtures made with these inferior aggregrates possess the required minimum stability, when properly designed in

solving stability problems in other divisions of soil mechanics, e.g., the selection of materials for, and the design of earth dams, embankments, foundations, etc. It should also be observed that if the maximum major principal stress, σ_1, supported in equilibrium by an element at any given point in a structure can be measured or calculated, and the c and ϕ values for the material at that point are

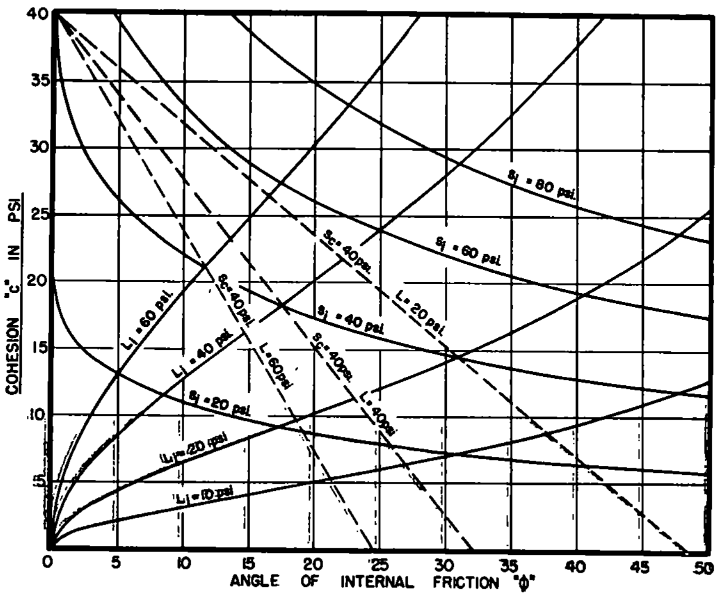

Figure 75. Relationships between s_l, L_l, s_c, L, c, and ϕ derived from the triaxial compression test

measured in the laboratory, the maximum minor principal stress, σ_{111}, acting on the element can be calculated by means of equations (11) or (23). When this information has been obtained, the normal and tangential stresses acting on any plane through the element can be easily calculated.

CONCLUSION

While the results outlined in this paper have been derived from the data obtained at airport locations extending right across Canada, it is realized that certain abnormal soils may occasionally exist here or in other countries to which these results may not directly apply. This may also be true of certain soils which would ordinarily be classified as normal. Therefore, it would be prudent to perform a certain amount of soil testing before applying these results elsewhere.

It is realized also that additional test data may lead to modification of some of the conclusions which have been expressed.

While on the basis of evidence obtained during the Department of Transport's investigation, this paper expresses some disagreement with the method for flexible pavement design currently advocated by the U. S. Corps of Engineers, which is considered to be unnecessarily conservative, it is a pleasure for the writer to pay tribute to the large amount of excellent investigational work that the U. S. Corps of Engineers is carrying on in many phases of the field of soil mechanics at the present time. This research will eventually greatly extend our knowledge of the engineering properties of soils, and of pavement design.

SUMMARY

1. While the ratio of load supported at one deflection to that supported at another deflection for a given bearing plate, appears to vary with the soil type from airport to airport, re-

presentative overall values for these ratios can be employed with reasonable accuracy when required.

2. The supporting capacity of a clay subgrade compacted to 95 percent of modified AASHO density was found to be appreciably greater than that of the uncompacted material at 85 percent of modified AASHO density. The lower 12-in. layer of compacted subgrade provided a greater increase in supporting value due to compaction than that given by an additional 12-in. compacted layer.

3. Field CBR and cone bearing ratings increase at a much faster rate due to field compaction of a clay subgrade, than the actual supporting value measured by a plate bearing test. The Housel penetrometer values deviate less in this respect, while the triaxial test results more nearly parallel the increase indicated by the plate bearing test.

4. The unit load supported on a 12-in. bearing plate is approximately twice that supported on a 30-in. plate at any given deflection over the range of 0.2 to 0.5 in. for cohesive subgrade soils.

5. Relationships between deflection, settlement, and elastic deformation have been developed for subgrades and flexible pavements on the basis of load test data.

6. A simple equation is presented for calculating the subgrade modulus at the surface of any given thickness of well compacted granular base course, if the subgrade modulus of the underlying subgrade has been measured.

7. Further data have indicated that the value of the base course constant K in the flexible pavement design equation $T = K \log (P/S)$ is dependent upon the size of bearing plate employed. This has necessitated the preparation of new charts for flexible pavement thickness requirements for various wheel loads for airports and highways.

8. The angle of pressure distribution through different thicknesses of granular base course for 30-in., and 12-in. bearing plates, for the flexible pavement design equation $T = K \log (P/S)$, is illustrated.

9. A method for the selection of base course materials and the design of flexible wearing course mixtures by means of the triaxial compression test is outlined.

ACKNOWLEDGEMENT

General administration of this investigation has been in the charge of Mr. F.C. Jewett, Chief, Wartime Construction, until his recent retirement, and of Mr. Theo. Ward, Assistant Chief, Wartime Construction. Since Mr. Jewett's retirement, general administration has been under Mr. Charles Flint, Superintendent Construction. In their respective districts, the program has been carried on with the generous co-operation of District Airway Engineers F. F. Cooke, John H. Curzon, Homer P. Keith, W. C. MacDonald, George W. Smith, and A. L. H. Somerville.

Arrangements were made to have the laboratory tests on all samples of base course and subgrade material obtained during the 1947 phase of this investigation, performed at the University of Alberta under the supervision of Dean R. M. Hardy.

In the preparation of the material upon which this paper is based, special mention should be made of the very able assistance provided by C. L. Perkins particularly, and by J. P. Walsh, D. S. Johnson, P. J. Prokopy, R. Applebaum, B. H. Newington, E. B. Wilkins, and D. Segal.

REFERENCES

1. Norman W. McLeod "Airport Runway Evaluation in Canada," Highway Research Board *Research Reports No. 4 B,* October (1947).
2. William S. Housel, "A Practical Method for the Selection of Foundations Based on Fundamental Research in Soil Mechanics", *Engineering Research Bulletin No. 13,* Department of Engineering Research, University of Michigan, Ann Arbor, Michigan.
3. D. P. Krynine, "Soil Mechanics", McGraw-Hill Book Company Inc., New York and London, p. 130 First Edition (1941).

4. L. W. Teller and Earl C. Sutherland, "The Structural Design of Concrete Pavements, Part 5", *Public Roads*, Volume 23, No. 8, April-May-June (1943).
5. Engineering Manual, Part XII, Chapter 2, U. S. Corps of Engineers, July (1946).
6. L. A. Palmer, Discussion following his paper "The Evaluation of Wheel Load Bearing Capacities of Flexible Types of Pavement", *Proceedings*, Highway Research Board Volume 26, (1946).
7. O. J. Porter, "Foundations for Flexible Pavements", *Proceedings*, Highway Research Board Volume 22 (1942).
8. E. B. Bail, "A Method for Estimating Required Thicknesses of Flexible Base", *Proceedings*, Highway Research Board, Volume 27 (1947).
9. W. G. Holtz, "The Use of the Maximum Principal Stress Ratio as the Failure Criterion in Evaluating Triaxial Shear Tests on Earth Materials", *Proceedings*, American Society for Testing Materials, Volume 47 (1947).
10. Phillip C. Rutledge, "Cooperative Triaxial Shear Research Program of the Corps of Engineers", Waterways Experiment Station, Vicksburg, Mississippi, April (1947).
11. L. W. Nijboer, "The Determination of the Plastic Properties of Bitumen Aggregate Mixtures and the Influence of Variations in the Composition of the Mix", *Proceedings*, The Association of Asphalt Paving Technologists, Volume 16, (1947).
12. Manual on Hot Mix Asphaltic Concrete Paving, The Asphalt Institute, New York, N. Y., U. S. A. (1945).
13. E. F. Kelley, "Application of the Results of Research to the Structural Design of Concrete Pavements", *Public Roads*, Volume 20, No. 5, July (1939).
14. V. A. Endersby, "The Mechanics of Granular and Granular-Plastic Materials with Special Reference to Bituminous Road Materials and Subsoils", *Proceedings*, American Society for Testing Materials, Volume 40, (1940).

DISCUSSION

W. K. Boyd[1]. In December 1946, Dr. Norman W. McLeod presented the results of an investigation of the principal airport runways in Canada before the Highway Research Board. Since the amount of data which he presented was considerable and the results were complex, it was not possible to present a discussion of his paper at that time. The paper has only recently become available for study, (this discussion was dated 12 Feb. 1948, ed.) and there was not sufficient time to digest the contents in its entirety. However, it is apparent that a wealth of very valuable information has been made available for study. relative to flexible pavement design. Doctor McLeod and his associates are to be congratulated for accumulating and presenting this information in such an able manner.

In the introductory portion of the paper it is stated that the principal motive for the study was the fact that it was felt the design criteria advanced by the engineering agencies in the United States were too conservative. In particular, it was believed the California Bearing Ratio (CBR) design procedure adopted by the Corps of Engineers resulted in the construction of runways

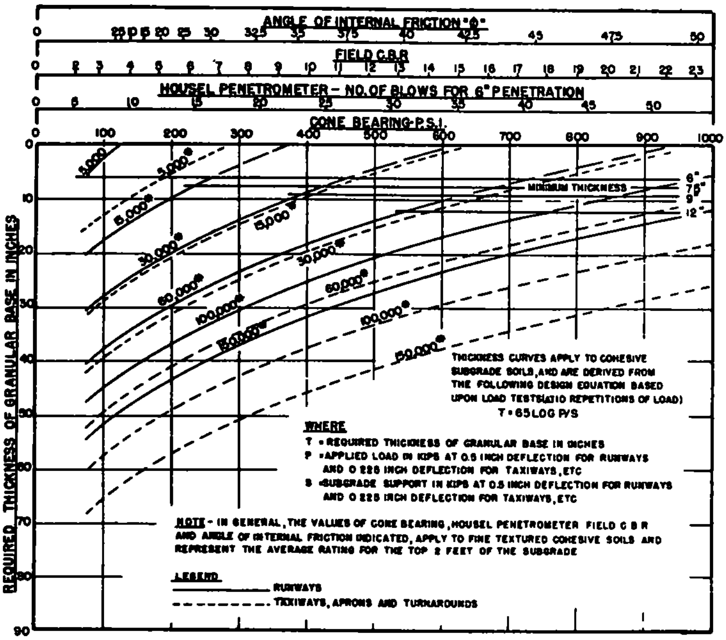

Figure A. Design Curves for Flexible Pavements for Runways and Taxiways, Etc., for Airplane Wheel Loadings (Full Load on Single Tire) - Cone Bearing, Etc. Tests

It can be expected that the data together with the analysis will constitute a real contribution to the store of knowledge with thicknesses of base and pavement considerably in excess of those actually required. The results of tests performed on the runways of Dorval airport at Montreal are cited to illustrate the alleged ultraconservative design requirements of the CBR test. It is believed the CBR method of design has been criti-

[1] Chief, Flexible Pavement Branch, Soils Division, Waterways Experiment Station, Mississippi River Commission, Corps of Engineers, Vicksburg, Mississippi.

cized more severely than appears justified. Therefore, this discussion will review Doctor McLeod's report primarily as it applies to comparisons with the CBR method of design as presently used by the Corps of Engineers.

As a result of the investigation which is reported, Doctor McLeod presents a new method of design for consideration. For the purpose of this review, Figure 97 from Doctor McLeod's paper which appears in Highway Research Board *Research Report No. 4 B* is reproduced as Figure A of this discussion. The thickness of granular base in inches is shown for both runways and taxiways, for wheel loads between 5,000 and 150,000 lb., and for a range of soil conditions from weak to strong. A valuable feature of the work performed by Doctor McLeod is the fact that the strength of the subgrade is evaluated by several methods: to wit, Cone Bearing, CBR, Housel penetrometer, triaxial, and plate bearing tests. Doctor McLeod has found that an approximate reasonable relationship exists so that although his design is based essentially on plate bearing tests it is possible to utilize other available tools. The Corps of Engineers also has determined that many of the empirical strength tests for subgrade soils can be correlated and confirms the relationship indicated between the cone bearing and CBR as being about 30 or 40 to 1. The Corps has had no experience with the Housel penetrometer. Doctor McLeod considers these design curves reasonably valid, providing the soil in-place and subjected to traffic is in the condition shown by one of the test methods. It should be noted that the requirements indicated for taxiways are considerably in excess of those for runways for any given wheel load and subgrade condition. It may be noted that for a runway design for a 5,000-lb. wheel load about 3 inches of total base and pavement are indicated as sufficient where the CBR is approximately 3 or the cone bearing value is about 100. If the taxiway curve is used, the requirements are about 12 inches. On Figure 100 of the referenced report highway curves are shown for 4,000; 7,000; and 12,000-lb. wheel loads. Although a 5,000-lb. load is not shown, it appears that highway and taxiway design curves are approximately equal. The highway design curve developed by the North Dakota Highway Department, based on some 1,300 cone bearing tests, more nearly conforms to Doctor McLeod's curves for taxiways rather than those for runways.

TABLE A

	15-Kip Wheel			
	Runway		Taxiway	
CBR	Can.	C.E.	Can.	C.E.
3	18	25	30	27
5	11	19	22	21
10	6[a]	12	9	14
	60-Kip Wheel			
	Runway		Taxiway	
CBR	Can.	C.E.	Can.	C.E.
3	38	45	50	50
5	31	33	42	37
10	18	21	29	23

[a] Recommended minimum.

The comparisons of the thickness requirements based on both the Canadian and the CBR method of design are shown on Table 1. The values for the thickness requirements based on the Canadian design are from Figure 1 while the CBR values are from the curves presented in Part XII, Chapter 2 of the *Engineering Manual*. Comparisons are shown for two wheel loads and for three CBR values. Looking first at the runway data, it can be seen that the CBR method is definitely more conservative than indicated by the Canadian curves. For a 15,000 lb. wheel load, this difference is marked. For the 60,000-lb. wheel load, the difference is not great for a CBR of 3 and may be considered identical for CBR values of 5 and 10. In the case of taxiways, the situation is somewhat reversed. Doctor McLeod indicates in nearly all cases that greater pavement thicknesses are required than are shown by the CBR curves. In the case of the 15,000-lb. wheel load, the differences are not great and are certainly no more than would be reasonably expected between independent investigators. In the case of the 60,000-lb. load for a CBR of 3, both require identical thicknesses of base. For CBR's of 5 and 10, the Canadian design requires greater thicknesses. It can be seen, therefore, that the two methods are not entirely inconsistent with each other and are not as different as one might be led to expect. The principal factor appears to be wide differentials in thickness requirements in the Canadian method between runways and taxiways. This differential is considerable, amounting to from 11 to 12 inches. It is conceded by all concerned that taxiways receive greater traffic concentrated over a narrower width. Also, planes are sometimes required to stand with motors idling on the taxiways waiting their turn to enter the runway. Some increase in the thickness requirements for taxiways as compared to runways may be reasonable. However, the experience of the Corps of Engineers does not indicate that such a large differential is justified on heavily used fields. The fact that the highway design curves prepared by Dr. McLeod compare with his taxiway curves strengthens the belief that the latter curves are more nearly correct.

It should be clearly understood that the CBR design curves are based on in-place data. A symposium relative to the development of the CBR method has recently been prepared and submitted to the American Society of Civil Engineers for publication. It is hoped that it will be printed in the near future. This symposium reviews the several test sections that have been constructed and subjected to accelerated traffic tests, the traffic tests performed on existing fields, and behavior studies from fields subjected to actual heavy wheel load traffic. In all cases, in-place CBR data and not soaked CBR values were obtained and used to verify the design curves.

The requirement that the samples be soaked is simply a method for evaluating the subject material in its most critical condition. It is probably true that subgrades in some locations and under certain conditions never become fully saturated. It should be clearly understood, however, that a soaked sample is not necessarily saturated (100 percent of the voids filled with water). The soaked condition implies that the sample has been completely immersed in water for four days. At the end of this period from 70 to 100 percent of the voids may be filled with water, the exact proportion being dependent on the type of soil, its density, and other factors. An investigation has been in progress for more than two years at the Waterways Experiment Station to gather factual data for in-place moisture content on typical fields. The results of this study demonstrate that under certain conditions the in-place moisture content will range between about 70 to 95 percent of all voids filled with water. As of the present time, this office has no satisfactory method for analyzing all factors such as soil type, density, drainage installation, rainfall intensity, level of water table, pavement cracking, and climatic conditions, and predicting the probable percent of saturation 10 or 20 years hence. Dr. McLeod repeatedly warns that the subgrade material must be evaluated on the basis of its weakest potential state. He neglects, however, to include in his report his method for preparing and/or adjusting the soil to the proper moisture and density which can be anticipated. It is expected that the problem will continue to be studied and the Corps will welcome any criteria which safely can supplant the admittedly conservative soaked condition.

W. K. Boyd and C. R. Foster[2]. The senior author presented a discussion of Part I of Dr. McLeod's paper at the 1947 meeting. Since then additional time has been available for the study of Part I and for a study of Part II which was presented by Dr. McLeod at the 1947 meeting. The statements made by the senior author relative to Part I are still considered generally applicable. The following dis-

[2] Chief and Assistant Chief, Flexible Pavement Branch, Soils Division, Waterways Experiment Station, CE, Vicksburg, Mississippi.

cussion pertains to both Parts I and II. (This section of the discussion was received September 9, 1948, ed.).

Dr. McLeod's paper "Airport Runway Evaluation in Canada, Part II" presents the results of tests on three additional fields together with additional analysis of previous data. Both the runway and taxiway thickness design curves presented in Part I have been revised. The changes in runway thicknesses have been negligible except for the heavier wheel loads where the thicknesses have been increased. Considerable revisions have been made to taxiway thicknesses to bring the taxiway thicknesses in better harmony with the runway thicknesses. It will be recalled that the wide difference between runway and taxiway thicknesses in the design curves was pointed out at the 1947 meeting.

Both Parts I and II of Dr. McLoed's paper deal essentially with two subject matters. For convenience the authors discuss them separately, although both subjects are treated in Part I and II of Dr. McLeod's paper. The first subject is a proposal for a method of designing the total thickness of base and pavement above the subgrade. The other subject is an outline for a possible method which may be useful in designing the bituminous mixes and base courses.

Total Thickness

Dr. McLeod's method of designing total thickness is based on the use of the plate bearing test. Essentially, he has correlated the results of the plate bearing tests with the capacity of the pavements to support airplane traffic on runways. This has been done by comparing the load in the plate bearing test at 0.5 in. deflection at 10 repetitions with the wheel load of the airplanes using the runways. Tests were made at 12 airfields where the pavements were adequate. For taxiways Dr. McLeod uses a different approach. He computes a "yield point deflection" by a method suggested by Professor Housel and determines that a value of 0.225 in. represents an average yield point. The design curves for taxiways are therefore based on the load in the plate bearing test at 0.225 in. deflection (revised to 0.35 in. in Part II) at 10 repetitions. This naturally results in greater thickness requirements for taxiways than for runways.

A large portion of the remainder of this part of the paper is taken up with an interesting study of the slope of the load-deflection curves obtained in the plate bearing tests and the correlations between the load at a given deflection in the plate bearing test and the CBR, North Dakota Cone Bearing, Housel penetrometer reading, and triaxial shear test results. The study results in relationships from which the subgrade support at 0.5 in. (0.35 in. for taxiways) can be determined from any of the tests mentioned above or from practically any type of plate bearing test.

From the study of the plate bearing test results Dr. McLeod develops the following relationships:

$$T = K \log P/S$$

Where T = Thickness in inches, K is a constant = 65 (revised later to increase with size of bearing plate), P is the applied load in kips, and S is the subgrade support in kips at a given deflection.

It can be seen that by varying the deflection at which S is obtained, any given thickness can be obtained for any given wheel load. Dr. McLeod has used 0.5 in. for runways and 0.225 in. (later 0.35 in.) for taxiways. With these values he has computed design curves for runways and taxiways which are fairly reasonable, especially since he has revised taxiway thicknesses. The thicknesses in general average a few inches less than the writers are accustomed to thinking of, but close agreement cannot be expected on this subject.

There is no denying that the data Dr. McLeod has collected plot up nicely and clear-cut relationships can be developed, especially for the study of the slope of the load-deflection curves. However, the nicety with which these relationships can be developed is not proof that the method of designing flexible pavements from plate bearing test results is applicable. The crux of the problem is how well the load at the selected deflection reflects the ability of the pavement to support traffic. Dr. McLeod states on page 18 of *Highway Research Board Report 4 B* "it was found that the lower quartile plate bearing value (the lower 25 percent point) at 0.5 in. deflection for 10 repetitions provided a load test value which appeared to be approximately equal to the maximum wheel load which the runways had been supporting under reasonably intensive traffic." Yet nowhere in the paper does he give a direct comparison of the load at 0.5 in. deflection and the maximum wheel load using the field. Further, information is not available in the report from which such a correlation can be made by the reader. Traffic data are given only for four fields in the 1947 paper, supplemented by additional data for one of the fields in the 1948 paper. Until proof is established that the load in the plate bearing test at 0.5 in. deflection, or for that matter at any specific deflection, represents the airplane wheel load which the pavement can carry, the proposed design method must remain a hypothesis.

Another criticism which the writers have is the fact that apparently all the runway data used for the correlation were from "satisfactory" pavements. The writers consider it necessary to have data from "failed" as well as "satisfactory" pavements before limits can be established. While at first glance it would appear that a design based on only "satisfactory" data would be conservative, this in not necessarily the case. Experience has shown that in any correlation there is a zone in which there are no failures, a zone in which there are all failures, and a zone in which there is an intermingling of both failures and satisfactory points. An example is Figure 1 of the paper "An Analysis of Wheel Load Limits as Related to Design," which appeared in Volume 22 of the *Proceedings*, Highway Research Board, 1942. This figure shows the correlation obtained between cone bearing, thickness, and traffic-carrying capacity. Approximately 40 percent of the "satisfactory" points are intermingled with "failure" points.

Another item which needs explanation is why the method of correlating load at a given deflection with carrying capacity, which was satisfactory for the runways, was not even attempted for the taxiways. While it is admitted that the loading conditions are not the same, still Dr. McLeod had the necessary elements. Surely, if the load at a given deflection in the plate bearing test represents the ability of a pavement to act as a taxiway, and Dr. McLeod assumes that it does, then a correlation with field behavior would not only be possible but logical.

In view of the fact that the papers have not substantiated the fact that the load at 0.5 in. deflection for 10 repetitions represents the load carrying capacity of the pavements, the writers take strenuous objection to Dr. McLeod's use of such terms as "actual supporting value as measured by the plate bearing test." The papers apparently are so well documented that the reader, after being subjected to the continued use of such statements, may subconsciously come to feel that the plate bearing test does measure the actual supporting value of the pavement, whereas such has not been proved. The plate bearing test does measure the supporting power of the pavement--to loading with a plate--but whether or not there is a relationship between this value and the pavement's ability to support airplane traffic remains to be proved.

One item for which Dr. McLeod should be complimented is the excellent information on density and moisture contents found at the airfields tested. As more of these data are accumulated the engineer can begin to know more about the conditions for which he is designing. It is very hard to design for an unknown quantity. The safest procedure is to take the limiting case, as the Corps of Engineers has, in assuming the "soaked" condition. Possibly a more reasonable approach than the "soaked" condition may be that taken by Wyoming and Colorado in their methods of weighting the thickness values for various soil types and conditions of rainfall and drainage. As additional data are accumulated on these items, engineers will be able to anticipate maximum moisture conditions with reasonable accuracy. As explained in the discussion given by the senior author in the 1947 meeting, the Corps of Engineers is continuing a study of the problem.

Bituminous Mixtures

The second part of the paper is devoted to bituminous mixes primarily. Dr. McLeod outlines a procedure for testing bituminous mixes with triaxial apparatus to determine if the mixes are satisfactory. It is suggested that the procedure is also applicable to the design of base courses. The procedure for applying the test results in an extension of that proposed by the Asphalt Institute, which is definitely empirical in spite of its use of the triaxial test. As used by the Asphalt Institute the results of the test are correlated with the behavior of the mixes under traffic and the Asphalt Institute has published a chart on which the results of the test can be plotted. One part of the plot is shaded and one part is not; if a point falls in the shaded area, the mix is unsatisfactory; if it falls in the open area, it is satisfactory. Dr. McLeod's paper goes into the theory of the test at some length. Throughout Part II of Dr. McLeod's paper examples are given to show the computations that could be made if values of the maximum lateral support could be determined. A value of 50 psi is assumed for the maximum lateral support in so many examples that the reader may get the impression that this is the value. The actual value of the maximum lateral support cannot be determined, as Dr. McLeod states near the end of the paper. Before the triaxial test can be used in a rational method of design, the test procedures must be adjusted so that the stress-strain relationships produced in the test simulate those produced by the airplane wheel load. This cannot be done at present for no one knows the stress-strain relationships produced by the wheel load. Until such is done, the use of the triaxial test is just another empirical method in which the test results are compared with the service behavior. For Dr. McLeod to infer that the proposed method "places the design of base courses and flexible wearing surfaces on a pounds per square inch basis" is misleading.

In summation, in reference to flexible pavements Dr. McLeod has presented (1) a method of design for thickness above the subgrade, and (2) a method of design for base and wearing courses. By the manner of presentation one would infer that these methods are new and that they are highly theoretical and consist of a much more rational approach to the problem than has heretofore been given. Actually, Dr. McLeod has only presented two empirical methods, the basic principles of which have been proposed before.

AUTHOR'S CLOSURE

by Norman W. McLeod

The discussions by Mr. Boyd, and by Mr. Boyd and Mr. Foster, touch on a number of very important considerations. Their comments are valued for the attention they have focused on these matters.

On the basis of the CBR test, Mr. Boyd presents in his Table A, a comparison of runway and taxiway thickness requirements as given by the Canadian Department of Transport and the U.S. Corps of Engineers' designs. Data for the Department of Transport design are taken from Figure 97 of our paper for the 1946 meeting published as Highway Research Board Research Report No. 4B in October 1947. Thickness requirements for the Corps of Engineers' design are taken from the CBR design curves in Part XII Chapter 2 of their Engineering Manual.

With reference to the Corps of Engineers' CBR design curves, Mr. Boyd states, "It should be clearly understood that the CBR design curves are based on in-place data". In the same paragraph, after referring to the field projects from which test data were obtained to check these design curves, he further explains, "In all cases, in-place CBR data and not soaked CBR values were obtained and used to verify the design curves". Nevertheless, in the next paragraph he adds the very significant statement, "The requirement that the samples be soaked is simply a method for evaluating the subject material in its most critical condition". Consequently, although he states that the Corps of Engineers' design curves were verified by in-place CBR tests, Mr. Boyd reiterates the well known requirement of their Engineering Manual that in-place CBR values must under no circumstances be employed for design, but that design must be based upon soaked CBR values.

Department of Transport design on the other hand has been correlated with in-place CBR values. Furthermore, our investigation of airports tested to date, has shown that the moisture contents of the top inch of subgrade samples soaked according to the procedure required for the Corps of Engineers' CBR test, has ranged all the way from 25 to 140 percent higher than the actual moisture contents found in the subgrade under runways that have been paved for several years.

Consequently, there is no justification for comparing the Department of Transport's and the Corps of Engineers' designs for flexible pavement thickness requirements on the basis of in-place CBR values as Mr. Boyd has done in his Table A. Such a comparison on the basis of the CBR test, is valid only if the Corps of Engineers' thickness requirements are given in terms of the soaked CBR values on which their design is based, and if data for the Department of Transport's design are expressed in terms of the in-place CBR values with which it has been correlated. Therefore, this comparison can be made only for airports for which the corresponding in-place CBR and soaked CBR ratings for the subgrade are available. This information was listed in Table 7 of our 1946 paper (Highway Research Board Research Report No. 4B) for a number of Canadian airports. It is repeated in Table 5 for several of these airports.

From the data of Table 5, a true comparison can be made between the Corps of Engineers' versus the Department of Transport design on the basis of the CBR test. This is provided in Table 6, using data from Figure 97 from our 1946 paper to represent Department of Transport design

TABLE 5

Average values for corresponding in-place CBR and soaked CBR ratings for the subgrades at several Canadian airports.

Airport	Average In-Place CBR	Average Soaked CBR
Lethbridge	12.6	4.6
Regina	7.3	3.3
Malton	6.6	3.5
Saskatoon	5.3	3.6
Dorval	3.9	3.1

and the thickness curves from the Engineering Manual for Corps of Engineers' design. It should be noted that Figure 97 from our 1946 paper no longer represents Department of Transport design because further analysis and additional data have led to the revisions that appear in Figure 33 of the present paper. However, the data of Figure 97 are utilized for the time being for Table 6, since Mr. Boyd's discussion was based on it.

Contrary to Mr. Boyd's conclusions, the data of Table 6 indicate that in not a single case for either taxiway or runway does the overall thickness required by Department of Transport design exceed or even equal that of the Corps of Engineers. Department of Transport taxiway thickness requirements for Dorval come nearest to those of the Corps of Engineers. In all cases the Corps of Engineers' design requirements greatly exceed those of the Department of Transport for runways, and this is generally true to a somewhat lesser degree for taxiways.

It is to be particularly noted, however, that for the reasons outlined in the present paper, Figure 97 of our 1946 paper no longer represents Department of Transport design for either runways or taxiways. It has been replaced by Figure 33

TABLE 6

Comparison of total thickness requirements between Department of Transport and Corps of Engineers methods for flexible pavement design. (Basis of comparison, Figure 97 of our 1946 paper and Corps of Engineers' CBR design curves).

15 Kip Wheel

Airport	Runway DOT	Runway CE	Taxiway DOT	Taxiway CE
Lethbridge	7.5*	20	9*	22
Regina	7.5*	23	15	26
Malton	7.5*	22	17	25
Saskatoon	10	22	21	25
Dorval	12	24	26	27

* Recommended Minimum Thickness

60 Kip Wheel

Airport	Runway DOT	Runway CE	Taxiway DOT	Taxiway CE
Lethbridge	14	35	24	39
Regina	24	42	35	47
Malton	26	41	37	45
Saskatoon	30	41	41	45
Dorval	34	44	46	49

of the 1947 paper. Consequently, to compare the differences in thickness requirements between the Corps of Engineers' and the revised Department of Transport design, the data of Table 7 should be used.

The data of Table 7 serve to emphasize the fact that the Corps of Engineers' method of design based upon the CBR rating of soaked subgrade samples, is very much more conservative for both runways and taxiways, than the design method of the Department of Transport. The data also stress the fact that the comparison made by Mr. Boyd in his Table A cannot be justified unless the Corps of Engineers abandon their present requirement that design must be based upon the CBR ratings of soaked subgrade samples, and establish their design on in-place CBR values instead.

When comparing the Corps of Engineers' and the Department of Transport's design methods, it should be particularly noted that for any one soaked CBR value, e.g.3, there can be a wide range of in-place CBR values, depending upon the moisture content that develops in the subgrade in different geographical areas. This is illustrated in a limited manner in Table 5. This means that for any given wheel load, the same design cannot be justified for a large number of airports merely because the subgrades happen to have identical soaked CBR ratings, as the Corps of Engineers currently insist in general, but that each airport should be designed with full consideration for all the existing local conditions.

TABLE 7

Comparison of total thickness requirements between current Department of Transport and Corps of Engineers' methods for flexible pavement design. (Basis of comparison, Figure 33 of present paper and Corps of Engineers' CBR design curves).

15 Kip Wheel

Airport	Runway DOT	Runway CE	Taxiway DOT	Taxiway CE
Lethbridge	7.5*	20	7.5*	22
Regina	7.5*	23	7.5	26
Malton	7.5*	22	9.0	25
Saskatoon	8.0	22	12.0	25
Dorval	12.0	24	16	27

* Recommended Minimum Thickness

60 Kip Wheel

Airport	Runway DOT	Runway CE	Taxiway DOT	Taxiway CE
Lethbridge	14	35	18	39
Regina	24	42	29	47
Malton	26	41	31	45
Saskatoon	30	41	35	45
Dorval	35	44	40	49

Near the end of his first discussion,

Mr. Boyd states that we have neglected to point out how we would adjust the moisture and density of a subgrade sample from a new site to the worst condition anticipated for it in service. It was thought that this has been done quite clearly in the section on "Results of Field Moisture and Density" in our 1946 paper. It is our belief that this information can be obtained in a reasonably satisfactory manner by studying the condition of the subgrade for projects in the same general area that have been paved for several years, provided that soil, drainage, topography, and other conditions are similar to those to be encountered at the new airport site.

Mr. Boyd and Mr. Foster have offered the criticism that nowhere in our papers for the 1946 and 1947 meetings has a direct comparison been given between the plate bearing value at 0.5 in. deflection for 10 repetitions for the lower quartile point and the maximum wheel load of the aircraft using the airfield. This statement is not quite correct, since the present paper provides such information for Dorval airport. The comparison suggested by Mr. Boyd and Mr. Foster is given in Table 8 for several other airports.

In Table 8, the third and fourth columns from the left list the maximum plate bearing values at 0.5 in. deflection for 10 repetitions, for unit loads of 55 and 85 psi respectively, which are the approximate tire pressures of the largest aircraft using the airport. Consequently, the data of the third and fourth columns from the left indicate the plate bearing values at unit pressures of 55 and 85 psi respectively, which correspond to the supporting values for a 30-in. diameter plate that are given in the second column.

It is to be emphasized that the data of Table 8 by themselves, which are for the weakest runway for each airfield, are of little value for the purpose Mr. Boyd and Mr. Foster have in mind, that is a comparison of plate bearing values at 0.5 in. deflection and 10 repetitions for the lower quartile point, versus the maximum wheel load using the airport. They are of little value in themselves, because the data must be weighted by other considerations, such as whether the maximum wheel load represents limited or capacity operations, whether the wheel load is for duals or a single tire, etc. Insofar as

TABLE 8

Comparison of plate bearing values at 0.5 in. deflection for 10 repetitions for the lower quartile point for the weakest runway versus the maximum wheel load of aircraft using the airport for several airports.

Airport	Total Load 30-Inch Plate 0.5 in. Deflection 10 Repetitions Weakest Runway Lbs.	Corresponding Supporting Value 0.5 Inch Deflection 10 Repetitions For Unit Load Of 55 psi. Lbs.	Corresponding Supporting Value 0.5 Inch Deflection 10 Repetitions For Unit Load Of 85 psi. Lbs.	Maximum Wheel Load for Aircraft Using the Airport Lbs.
Moncton	54,500	-	48,900	45,000
Winnipeg	40,000	-	25,700	37,000
Lethbridge	53,600	-	47,600	37,000
Saskatoon No. 1	22,000	11,700	-	12,500
Grande Prairie	33,400	28,200	-	12,500
Fort St. John	27,400	18,600	-	12.500

Canadian airports are concerned, it is further complicated by the fact that runways also serve as taxiways to a very considerable degree.

To discuss the data of Table 8 for each of the airports listed with all these considerations in mind, would require much more space than is available for this closure. There is a full discussion of this matter in connection with Dorval Airport at Montreal, under the section heading, "Thickness Design Charts for Flexible Pavements for Airports", in this paper. It is clearly demonstrated that insofar as the runways at Dorval are concerned, the Department of Transport method of design is conservative.

It might be added that the runways at the airports listed in Table 8 have been carrying the wheel loads shown for a number of years, without signs of distress. It should be particularly noted that in the case of Winnipeg, and Saskatoon No. 1 airports, the plate bearing values for the unit loads equal to the tire pressures of the heaviest airplanes using them, are less than the wheel loads of these aircraft. This is a further indication that even the Department of Transport's plate bearing method for runway design may be relatively conservative rather than otherwise.

Mr. Boyd and Mr. Foster point out the desireability of including "failed" as well as "satisfactory" runways when working out a method for flexible pavement design. This suggestion would seem to have considerable merit provided all the factors that may have contributed to "failure" are properly weighted in each case. However, there have been extremely few runway failures at the several hundred airports built in Canada, and where failure has occurred it has nearly always been due to disregard for some well established design or construction principle, rather than to overloading. Consequently, we do not have sufficient data for "failed" and "satisfactory" runways for preparing the type of diagram that Mr. Boyd and Mr. Foster have in mind.

Mr. Boyd and Mr. Foster ask "Why the method of correlating load at a given deflection with carrying capacity, which was satisfactory for the runways, was not even attempted for the taxiways", and suggest that we "had the necessary elements" for such a correlation. The answer is, that at this time we do not have the necessary data. At most Canadian airports the runways have also served as taxiways to a very considerable degree, and up to the present the taxiway systems have been very much less extensive than those at many United States airports. To date the Department of Transport's airport evaluation program has been confined very largely to runways. Experience has shown that a greater thickness is required for taxiways, aprons, and the turnaround areas at the ends of the runways, than for the runways themselves. This added thickness can be logically obtained by basing taxiway, apron, and turnaround design on a smaller critical deflection than the 0.5 in. deflection employed for runway design. Since the average yield point deflection for all plate bearing tests on flexible pavements was found to occur at 0.35 in., this value has been arbitrarily assumed as the critical deflection for the design of taxiways, aprons, and turnarounds, for the reasons outlined in the paper itself. If additional experience indicates that this critical deflection is either too large or too small, it will be modified accordingly.

Mr. Boyd and Mr. Foster state that "The plate bearing test does measure the supporting power of the pavement - to loading with a plate - but whether or not there is a relationship between this value and the pavement's ability to support airplane traffic, remains to be proved". In spite of the published results of the comprehensive investigations that the Corps of Engineers have made, there are a great many informed engineers who feel that a parallel criticism could be made with regard to the Corps of Engineers' use of the CBR test for flexible pavement design. A similar statement could be made concerning every method for flexible pavement design that has been proposed up to this time.

It is true that no definite, final, relationship has been established so far between plate loads and tire loadings on flexible pavements, due to the experimental difficulties involved in any formal investigation of this problem. It would seem that the most satisfactory practical approach to this matter can be made by relating the plate bearing data for a great many airports with the wheel loads of the aircraft using them. It is this approach that has been adopted for the Department of Transport's testing program. On the basis of load test and traffic information obtained up to this time, it is believed that load test data for rigid bearing plates at 0.5 in. deflection provide a reasonable criterion for the design of flexible pavements for runways. As pointed out earlier in this discussion, this approach seems to even provide conservative thickness values. It is realized that further investigation will undoubtedly lead to modification of the Department of Transport's present method of design. We are clearly aware of the nature of several additional refinements that should be made if the necessary test data were available. Nevertheless, until the information required for these refinements is developed through further investigation, it is our belief, based upon our own experience, that a reasonable design will result from the use of the Department of Transport's present method. It should be observed that a number of other organizations, including the Bureau of Yards and Docks of the U. S. Navy, utilize the plate bearing test for the evaluation and design of flexible pavements. Consequently, the use of plate bearing tests for flexible pavement design would appear to be at least as far beyond the "hypothesis" stage, as any other test in use at the present time.

Referring to the section of our paper dealing with the design of base courses and bituminous mixtures by means of the triaxial test, Mr. Boyd and Mr. Foster state that the procedure described is an extension of the method proposed by the Asphalt Institute. We believe that upon closer examination, Mr. Boyd and Mr. Foster will find that this statement is not correct. The Asphalt Institute's triaxial chart resulted partly from a study of empirical data, and partly from considerations based upon the theory of elasticity. The curved lower boundary of the Asphalt Institute diagram was established entirely on the basis of the theory of elasticity, for an applied vertical load of 100 psi.

The method for bituminous paving mixture design, based upon the triaxial test, that is outlined in our paper, on the other hand, is based throughout upon the geometrical and trigonometrical implications of a straight line Mohr envelope in the usual Mohr diagram. Consequently, the two methods, while both utilizing the triaxial test, approach the problem of the design of bituminous mixtures on the basis of entirely different concepts.

As Mr. Boyd and Mr. Foster point out, the amount of the maximum lateral support that can be mobilized to oppose an applied vertical load on bituminous surfaces, base courses, or soils, is a matter on which there is very little existing information. We were careful to emphasize this point in our paper. The value of 50 psi for lateral support L, which was employed for the various sample calculations in the latter part of the present paper, was selected for illustrative purposes only. It was not intended that this value should be considered to represent the maximum amount of lateral support always available.

In general, the maximum lateral support that can be developed probably varies from project to project, and particularly from material to material. In the case of bituminous mixtures, it seems quite likely that the maximum lateral support available depends upon the properties of each paving mixture. In connection with Figure 64 of the present paper, it was demonstrated that the amount of lateral support available in a bituminous pavement will probably always be greater than twice the cohesion c of the bituminous mixture. In Figure 65 sample calculations for the design of a bituminous paving mixture on this basis are illustrated.

It is rather likely that the assumption that $L = 2c$, which was employed in connection with Figure 65, represents a much too conservative approach to bituminous pavement design, since Figure 64 indicates that the amount of lateral support L that can be developed, is likely to exceed $2c$, in general.

From Figure 64 (d) and equation (23), it would appear that the maximum lateral support L available for a bituminous pavement is more likely to be

$$L = 2c\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}} \tag{31}$$

That is, the maximum amount of lateral support L that can be developed by a bituminous pavement, is equal to the unconfined compressive strength of the bituminous mixture as obtained from its corresponding Mohr diagram, e.g. Figure 64 (d). Since the weight of element (2) of Figure 64 (a) has been neglected in this development, it is not unlikely that even equation (31) provides conservative values of lateral support L for the design of bituminous mixtures.

In Figure 76, sample calculations are illustrated for the design of a bituminous mixture on the basis that the maximum lateral support L available is given by equation (31). These calculations indicate that a bituminous mixture with cohesion $c = 10$ psi and an angle of internal friction $\phi = 28$ deg., would provide a stable pavement if the applied vertical load were 80 psi, but that the pavement would be unstable for a vertical load of 100 psi.

Mr. Boyd and Mr. Foster state that "--the use of the triaxial test is just another empirical method in which the test results are compared with service behavior. For Dr. McLeod to infer that the proposed method places the design of base courses and flexible wearing surfaces on a pounds per square inch basis is misleading". On the basis of the evidence provided in the paper itself, and in the present closure, we can only express our complete disagreement with Mr. Boyd's and Mr. Foster's remarks, since the method we have outlined is not empirical, and it very definitely

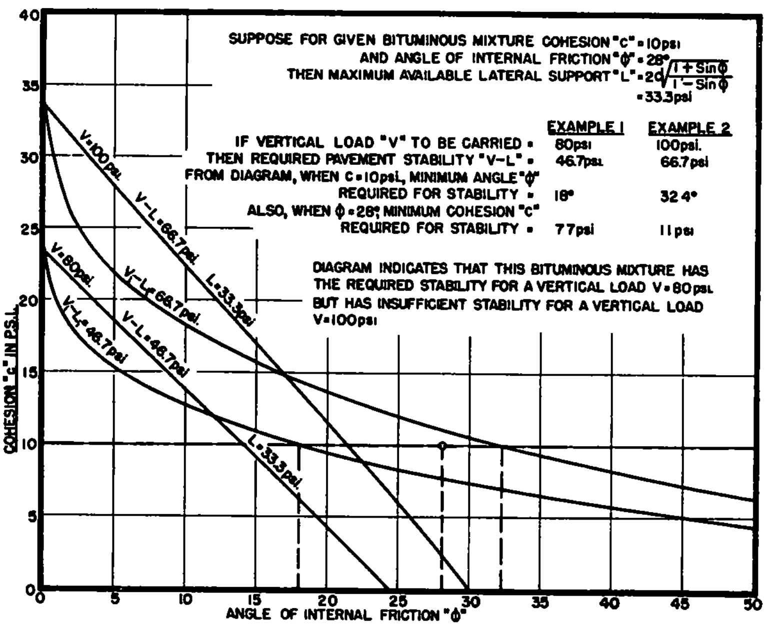

Figure 76. An example of Bituminous Mixture Design by the Triaxial Method when Lateral Support L assumed equal to $2c\sqrt{\frac{1 + \sin\phi}{1 - \sin\phi}}$

places the design of bituminous mixtures on a pounds per square inch basis. The method we have described cannot be used on any other than a pounds per square inch or similar unit load basis, since this feature is a fundamental part of it.

We are quite well aware that further refinements in this approach to the design of bituminous mixtures should be made as soon as the necessary information becomes available. For example, more accurate information should be obtained concerning the maximum lateral support that can be developed. It is also true that the method for running the triaxial test should be correlated with the rate of strain to which the paving mixture is subjected in the field. Some special consideration may have to be given to the viscous resistance which these mixtures can develop under moving loads, to the temperature of test, the size of specimen, etc. These however, are details and even though some of them may be important details, they do not alter the fundamental fact that the approach based upon the triaxial test which is outlined in this paper, makes it possible to design the strength or stability of bituminous wearing surfaces on a pounds per square inch or other unit load basis.

It should be the goal of engineers in every field to evaluate the strength of the materials they are using on a pounds per square inch basis. This is standard practice with structural design, in the design of footings, in rigid pavement design, etc. Consequently, it is our firm belief that there could be no greater deterrent to progress in the design of bituminous mixtures, than a tendency on the part of highway and airport engineers to continue with the use of the various currently employed indicator tests, for attempting to evaluate the stability of these mixtures. While in the absence of more fundamental methods, they have served a useful purpose in the past, engineers should be thoroughly aware of the fact that these indicator tests are not basic in nature, and are actually little more than rough rule of thumb methods. Among the indicator tests in most common use on this continent at the present time are various extrusion tests of the Hubbard Field stability type, unconfined compression tests of which the Marshall test is the most recent version, and with these there must also be included the percentage stability values given by the Hveem stabilometer. All of these methods are mere indicator tests because they do not provide data that can be analysed in terms of shear, or flexural strength, etc., on a pounds per square inch basis.

We cannot hope to make any reasonable further progress in the design of bituminous paving mixtures, until these various indicator tests are replaced by a more fundamental method. The triaxial test seems to have the necessary qualifications, since from its use the individual contributions to the total stability or strength, made by the cohesion factor, the internal friction factor, and even the viscous resistance factor if required, can each be evaluated on a psi basis.

Mr. Boyd and Mr. Foster make several statements in their final paragraph which do not seem to be entirely in accord with the facts. Referring to the method of design for thickness above the subgrade, and the method of design for base course and bituminous mixtures described in our paper for the 1946 meeting and the present paper, they state "By the manner of presentation one would infer that these methods are new and that they are highly theoretical and consist of a much more rational approach to the problem than has heretofore been given. Actually Dr. Mc Leod has only presented two empirical methods, the basic principles of which have been proposed before."

The two sentences just quoted from the final paragraph of the discussion by Mr. Boyd and Mr. Foster, do not seem to be consistent with the first sentence of the third paragraph of Mr. Boyd's initial discussion, wherein he states, "As a result of the investigation which is reported, Dr. McLeod presents a new method for consideration". The underlining has been added.

We cannot agree that the method for bituminous mixture design based upon the triaxial test, which has been outlined in our papers is an empirical method. Its derivation is based throughout on the mathematical implications of a straight line Mohr envelope. Consequently, this method is definitely not empirical, but is entirely theoretical and rational in its approach. We have carefully avoided specific claims to anything new at any point in these two papers, since it frequently happens when an investigator claims a new discovery that a search may reveal that Noah had a complete or at least a partial description on file in his library on the Ark, or that it has been recorded elsewhere since the Flood. Nevertheless we would be interested in having the references which indicate that the specific approach to the design of bituminous mixtures which has been outlined in the present paper, "has been proposed before".

Concerning the method described in our paper for determining the thickness of material required above the subgrade, it is difficult to see on what basis Mr. Boyd and Mr. Foster can justify the statement that "by the manner of presentation one would infer" that this method is "highly theoretical". It should be clear to everyone who has read our papers for the 1946 and 1947 meetings, that our approach to this problem has been entirely empirical. From first to last it has been governed completely by the analysis of experimental data. Our entire approach to this matter is clearly and carefully expressed in the following paragraph which appears on Page 4 of our paper for the 1946 annual meeting as published in the Highway Research Board Research Report No. 4B. "It is emphasized that in starting this program of runway testing, the Department of Transport had no theories of its own to either prove or disprove. The principal objective was to obtain the necessary data, and let this information speak for itself. This principle has been followed consistently throughout the entire investigation."

NATIONAL RESEARCH COUNCIL

The National Research Council is a cooperative organization of the scientific men of America. Its members include, however, not only scientific and technical men but also business men interested in engineering and industry. It was established in 1916 by the National Academy of Sciences.

The Charter of the National Academy of Sciences, under a special act of the Congress approved by President Lincoln in 1863, provides that "the Academy shall, whenever called upon by any Department of the Government, investigate, examine, experiment, and report upon any subject of science or art."

DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

OFFICERS AND EXECUTIVE COMMITTEE

Chairman..FREDERICK M. FEIKER

Vice-Chairman..HAROLD K. WORK

Executive Secretary..LOUIS JORDAN

LYMAN J. BRIGGS, WILLIAM B. KOUWENHOVEN, THOMAS H. MACDONALD

HIGHWAY RESEARCH BOARD

The Highway Research Board is organized under the auspices of the Division of Engineering and Industrial Research of the National Research Council. Its purpose is to provide a national clearing house for highway research activities and information. The membership consists of 36 technical, educational, industrial, and governmental organizations of national scope. Associates of the Board are firms, corporations, and individuals who are interested in highway research and who desire to further its work.

The Board provides a forum for the discussion and publication of the results obtained by individual research workers; organizes committees of experts to plan and suggest research work and to study and correlate the results; publishes and otherwise disseminates information; provides a research information service; and carries on fact finding investigations. With the cooperation of the highway departments of the states and Territories and the Public Roads Administration, the Highway Research Board conducts a Highway Research Correlation Service, to aid the many highway research agencies to correlate their work through personal visits, conferences, committee work, and dissemination of pertinent information.

HIGHWA
RESEARC
BOARD

RESEAR
REPOR
3-15

NATIONAL
RESEARC
COUNCIL