A CANADIAN INVESTIGATION OF FLEXIBLE PAVEMENT DESIGN

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Introduction.

During the past fifty years, a great many different equations and methods of design have been proposed by individuals and organizations for the thicknesses of flexible pavement required to carry wheel loads of various magnitudes, when supported by subgrades with a wide range of bearing capacities. Some of these designs have been derived from theoretical soil mechanics, some have been developed from partly theoretical and partly empirical considerations, and others have been based entirely upon empirical data.

The methods of design for determining thicknesses of flexible pavements for airport runways, that are best known among engineers in North America, are probably those recommended by the U.S. Corps of Engineers[1], Civil Aeronautics Administration[2], U.S. Navy[3], and Public Roads Administration[4] For highway construction on the other hand, it would appear that no method of flexible pavement design so far proposed has been widely accepted, although several seem to have acquired regional recognition. It is to be noted that no method of design for thickness of flexible pavements which has been suggested so far, even begins to approach the general acceptance accorded by both highway and airport engineers to the Westergaard method for rigid pavement design.

Because of the enormous number of airports that were required for training purposes, for global air transport, and for the various theatres of operations during the world war just concluded, the method of design for thickness of flexible pavement which has received the greatest attention on this continent, and probably among engineers throughout the entire world, is that which was developed by the Corps of Engineers of the U.S. War Department, on the basis of the C.B.R. (California Bearing Ratio) rating of soaked subgrade samples. It is also undoubtedly true, that this method of design has received a greater amount of field and laboratory study than that proposed by any other individual or organization. As a result of their many investigations, the U.S. Corps of Engineers believe that their method of design has been amply confirmed by experimental data.

In spite of this imposing array of evidence, the principal engineers of Canada's Department of Transport, which with very few exceptions has been responsible for all Canadian airport construction, have for some time been firmly convinced that some of the pavement design procedures for airport runway construction being currently advocated in the U.S.A., are unnecessarily conservative. They believe also, that they have sufficient traffic data in connection with airport runways in all parts of Canada, on which to base a reasonable opinion concerning the adequacy or otherwise of any proposed method for flexible pavement design.

Canada's busiest airport, Dorval, is located near the city of Montreal. It is one of the terminals for air transport between North America and Europe, and was used quite intensively during the war for the ferrying of 4-motored aircraft from this continent to Britain.

The average overall thickness of flexible surface, base course, and sub-base at Dorval is about 14 inches. The clay subgrade has a C.B.R. rating of 3, after the samples have been subjected to the standard soaking test. In winter, the frost penetration is several feet.

Based upon this information, the aircraft wheel loadings which should not be exceeded for capacity operations at Dorval according to the design requirements of the U.S.E.D., C.A.A., and P.R.A., are as follows:

According to U.S.E.D.[1]	design	-	5,000	pounds	
" " C.A.A.[2]	"	-	7,500	"	
" " P.R.A.[3]	"	-	10,000	"	(maximum).

Actual traffic data for Dorval over the period from January 1, 1942 until January 31, 1947, for operations by aircraft of the gross loadings indicated, were as follows, (each take-off or each landing is counted as one operation):

Over 211,000 operations of aircraft weighing 25,000 pounds or more

Over 89,000 operations of aircraft weighing 50,000 pounds or more

Over 23,000 operations of aircraft weighing 65,000 pounds or more

In one day last fall there were 77 operations by Lockheed Constellations, which weigh from 80,000 to 90,000 pounds.

The field C.B.R. value (field condition and unsoaked) for the subgrade under the runways at Dorval, ranges from 2.7 to 4.9, and averages 3.9.

The District Airway Engineer at Montreal, Mr. John Curzon, reports that at no time since the airport went into operation during the winter of 1941-42, has traffic been delayed because of poor runway condition, even during the spring break-up.

If the runways at Dorval had been designed on the basis of the soaked C.B.R. rating of the subgrade, the U.S.E.D. design chart indicates that an overall thickness of sub-base, base course, and wearing surface, of approximately 30 to 35 inches would have been required to support the wheel loadings which it has been carrying for several years with its present thickness of about 14 inches.

The experience of the Department of Transport at Dorval can be verified by that at many other airports in Canada. In Table I below, traffic data and certain essential descriptive characteristics are summarized for Malton Airport at Toronto, Stevenson Field at Winnipeg, and the airport at Lethbridge, Alberta, which are among Canada's busier airfields.

Table I

Traffic Data for Toronto, Winnipeg and Lethbridge Airports.

January 1, 1941 to January 31, 1947

Airport	Overall Thickness Pavement and Base Inches	Average C.B.R. Value Soaked Subgrade Samples	Wheel Load Rating U.S.E.D. Design Curves	Actual Traffic Data to Nearest Full Thousand Number of Operations of Aircraft Weighing More Than 7,500 lbs.	15,000 lbs.	25,000 lbs.	50,000 lbs.	64,000 lbs.
Toronto	8 to 10	3.5	2000 lbs. (approx.)	299,000	79,000	41,000	3,400	3,000
Winnipeg	(8"-2 rwys. (14"-1 rwy.	3.3	2000 lbs.) 5000 lbs.)	319,000	96,000	25,000	several hundred	
*Lethbridge	6 to 8	4.6	2000 lbs. (approx.)	229,000	35,000	4,400	several hundred	

*Traffic data for the period of January 1, 1942, to January 31, 1947.

Table I indicates that the runways at Toronto, Winnipeg, and Lethbridge have been supporting wheel loads which exceed by several times their rated safe carrying capacity according to some current U.S. designs. This also applies to a great many other Canadian airports where the runways have been constructed on clay or clay loam subgrades.

As a result of this experience, engineers of the Department of Transport believe that the relatively thin bases and wearing surfaces on runways at most Canadian airports, have considerably greater load carrying capacity than their rating according to several current U.S. designs would indicate. In particular, it is felt that a design based upon the C.B.R. rating of soaked subgrade samples could not ordinarily be justified for airport runway construction in Canada.

It was because of the necessity for developing a method of design of their own, that the principal engineers of the Department of Transport arranged to have the current investigation undertaken in the early spring of 1945.

The objectives of this investigation were:

(1) To determine the load carrying capacity of existing runways, by means of plate bearing tests (repetitive).

(2) To obtain information that could be employed for the design of either rigid or flexible pavements, by means of repetitive load tests on subgrade, base course, and surface.

(3) To ascertain the field moisture content and density of the base course and subgrade at each test location.

(4) To conduct certain simple field tests on the subgrade, such as cone bearing, Housel penetrometer, and C.B.R., which might be correlated with the results of the cumbersome and costly plate bearing test.

(5) To secure large undisturbed samples of base courses, subbase, and subgrade, on which the usual physical tests, mechanical analysis, and compaction tests could be made, and undisturbed samples of the subgrade for C.B.R. (both field and soaked conditions), triaxial compression, shear, and consolidation tests.

(6) To prepare soil maps for each airport, based upon the pedological system of soil classification, and to correlate soil type with load test data, if possible.

(7) Upon the basis of plate bearing repetitive load test data, to establish a design equation or set of curves for required thickness, which could be employed with reasonable confidence for the design of flexible pavements to support aeroplane wheel loadings of any magnitude.

2. Location and Brief Description of the Airport Projects Investigated.

The locations of the ten airports which have been included in the investigation so far are shown in Fig. 1. Their geographical distribution covers a wide area extending from Eastern Canada to the southeastern approaches to Alaska.

Fort Nelson, Fort St. John, and Grande Prairie, are part of the North West Staging Route, and were built during the war for the ferrying of aircraft, personnel, and supplies to Alaska and beyond during the war. The other seven airports have been regular ports of call on the scheduled route of Trans-Canada Airlines for some years.

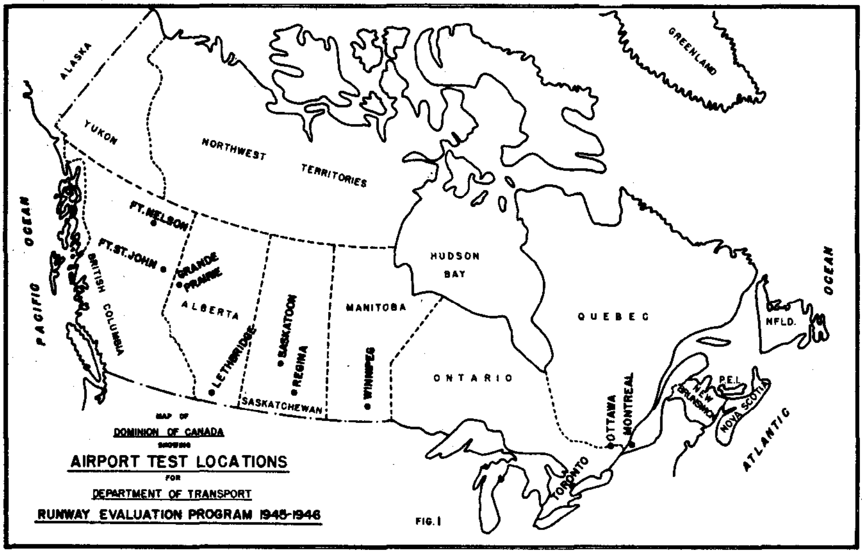


FIG. I

Since all of the runways at these airports had been constructed for at least one year, and generally for several years before the testing program began in the early spring of 1945, it could be reasonably assumed that the subgrade, sub-base, and base course, had attained approximate equilibrium insofar as the distribution of soil moisture was concerned.

Table 2 contains a general description of the subgrade, sub-base, base course, and wearing surface for each of the ten airports. At Uplands Airport at Ottawa, the subgrade consists of about 80 feet of clean sand, and at Fort Nelson there is from 3 to 5 feet of sand over clay. The subgrades at the other eight airports are composed of clay or clay loam, with C.B.R. values (soaked) varying from 2 to 4.5.

The runways at the ten airports tested so far have flexible pavements. The design for rigid pavements has received a great deal of study over the years, and it seemed unlikely that an investigation of our own would add anything worth while to the very fine analysis and method of design which has been worked out by Westergaard for this type of pavement. Flexible pavement design on the other hand, has until quite recently received very little fundamental study, probably because of the inherent difficulties involved.

Table 2

General Description of Airport Sites

Airfield	Depth to Water Table	Pavement	Base Course	Sub-Base	Subgrade P.R.A. Classification	Subgrade L.L. Ave.	Subgrade P.I. Ave.
Ft. St. John	Deep	3.5 to 6"- R.C.4 & 150/180 Pen. Bituminous Mixture	5.5 to 10" Crusher Run Gravel	5.0 to 17.0" Pit Run Gravel	A-7	49.2	27.0
Grande Prairie	Deep	2.0 to 8.0"- S.C.5 Bituminous Mixture	6.5 to 16.5" Mechanical Stabilization and Gravel	None	A-7	63.9	38.5
Saskatoon	Deep	1.5 to 3.5"- S.C.5 Bituminous Mixture	4.5 to 6.5" Gravel	None	A-7 A-6	46.5	23.7
Lethbridge	Deep	1.5 to 3.5" S.C.5 Bituminous Mixture & Surface Treatment	4.0 to 7.5" Gravel	None	A-7 A-6	39.5	20.0
Dorval (Montreal)	4.0' to 6.0'	4.0 to 6.0" Pen.Macadam with Sheet Asphalt Top	3.0 to 5.0" Water Bound Macadam	3.0 to 9.0" Pit Run Gravel	A-4 A-7 A-6	33.7	13.4
Winnipeg	Deep	3.0 to 4.0" S.C.5 Bituminous Mixture	5.0 to 10.5" Mechanical Stabilization	None	A-7	64.4	36.7
Malton (Toronto)	Deep	3.5 to 9.0" S.C.5 Bituminous Mixture	1.5 to 7.0" Gravel	None	A-4 A-7 A-6	32.1	13.8

Table 2 (Cont'd.)

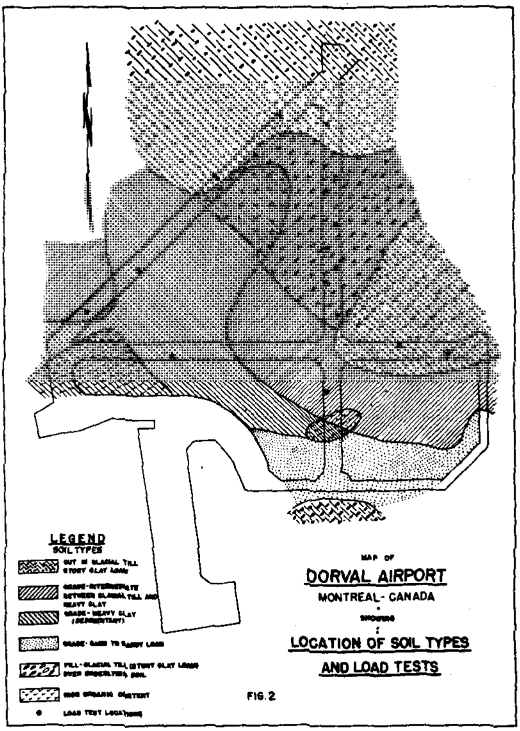
Airfield	Depth to Water Table	Pavement	Base Course	Sub-Base	Subgrade P.R.A. Classification	Subgrade L.L. Ave.	Subgrade P.I. Ave.
Uplands (Ottawa)	Deep	2.0 - 3.0" S.C.5 Bituminous Mixture	2.3 to 6.0" Gravel	None	A-2	18.8	0
Ft. Nelson	2' to 3'	4.5 to 5.5 150/180 Pen. Bituminous Mixture	6.0 to 10.0" Pit Run Gravel	None	4' to 5' of Sand over Clay	24.8 (for clay)	10.4 (for clay)
Regina	Deep	0.5 to 1.0" Surface Treatment	5.0 to 7.0" Mechanical Stabilization	None	A-7	72	38

Except where specifically indicated to be otherwise, this entire paper deals with the test data obtained for the eight airports with clay subgrades. Arriving at a reasonably satisfactory design for runways to be placed on granular subgrade soils, is in general not a too difficult problem. It is for clay subgrades that the greatest thicknesses of base and surface are required, and it is in connection with clay subgrades that the greatest difference of opinion exists at the present time concerning the thickness of flexible pavement that should be selected.

3. Soil Survey and Pedological Soil Maps for Airport Sites.

Pedological soil surveys were conducted at each airport site by qualified soil surveyors provided through the courtesy of the Central Experimental Farm at Ottawa, and the Soils Department of the University of Saskatchewan. From the soil surveys, a pedological soil map of each site was prepared, showing the area occupied by each soil type, Fig. 2.

In general, not more than one or two principal soil types occurred at each airport site, and most frequently there was only one. Fig. 2 indicates the areas occupied by the two main soil types at Dorval, one a fluvial deposit laid down by the St. Lawrence River which flows nearby, and the other being composed of boulder clay or glacial till left by the ice ages. The remainder of the site consists chiefly of soil which is transitional between the two principal types, or of a layer of boulder clay deposited over the



fluvial or transitional soil types during construction operations. Small areas of sand and muck soils also occur.

In Table 3, a comparison is made between values of subgrade support under the pavement measured at 0.5 inch deflection for the portions of the runways on the fluvial and glacial soils.

It is apparent from Table 3 that the boulder clay at Dorval has about twice the subgrade bearing capacity of the fluvial soil.

Table 3

Comparison of subgrade support at 0.5 inch deflection for fluvial and glacial till soils at Dorval airport.

	Subgrade support in lbs. at 0.5" deflection. 30-inch diameter bearing plate.		
No. of Repetitions of Load	Fluvial Soil A-4 to A-7	Glacial Till A-4 to A-7	
	Grade	Cut	Embankment
1	15000	29000	28000
10	13000	24000	23000
100	12000	21000	20000
1000	11500	19000	19000
10000	11000	17000	18000

While both glacial and fluvial soils fall into the same range of P.R.A. classification, the glacial soil contains an appreciable percentage of fine gravel. The average plasticity index of the glacial soil was about 11, and the average P.I. of the fluvial soil was about 20.

The pedological soil survey furnishes valuable information to airport and highway engineers, by indicating the areas occupied by soils with different engineering properties.

4. Evaluation of Field Moisture Content and Density Tests.

During the past four or five years, a number of technical articles have suggested that all subgrades and base courses are likely to become saturated with time, and that designs for pavement thickness should be based upon this anticipated condition.

One objective of the Department of Transport's investigation, consisted of determining the in-place moisture contents and densities of the base course, and of each 6-inch layer of the subgrade to a depth of 18 inches, and frequently to 24 inches below the surface of the subgrade. Large samples were taken from each of these layers and sent to the laboratory for various tests, including modified A.A.S.H.O. compaction. From the test data obtained in place in the field, and on samples sent to the laboratory, the degree of saturation in place can be determined. In addition, the field moisture content of the subgrade can be expressed as a percentage of the plastic limit, and of the modified A.A.S.H.O. optimum moisture. The various relationships obtained from this information are summarized in Figs. 3 to 5.

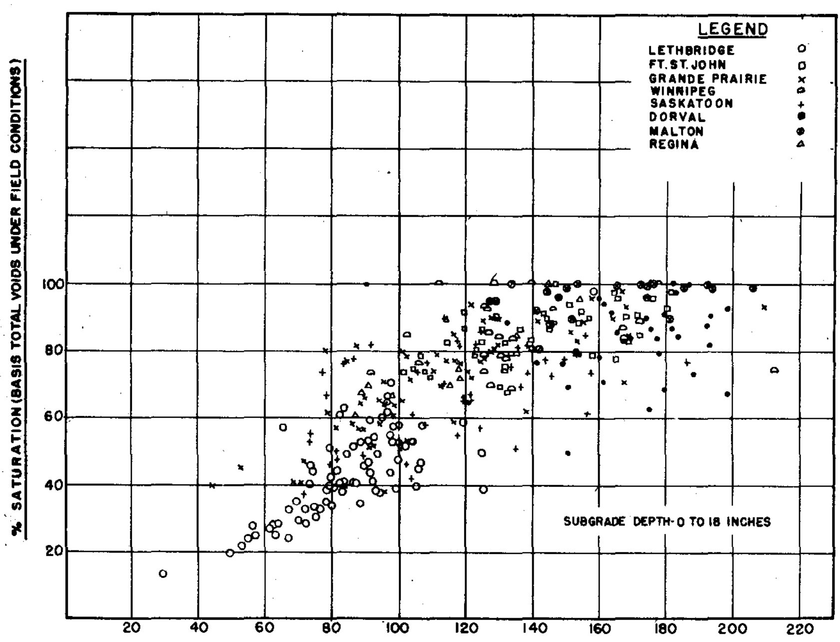


FIG. 3 % SATURATION VERSUS FIELD MOISTURE AS % OPTIMUM MOISTURE

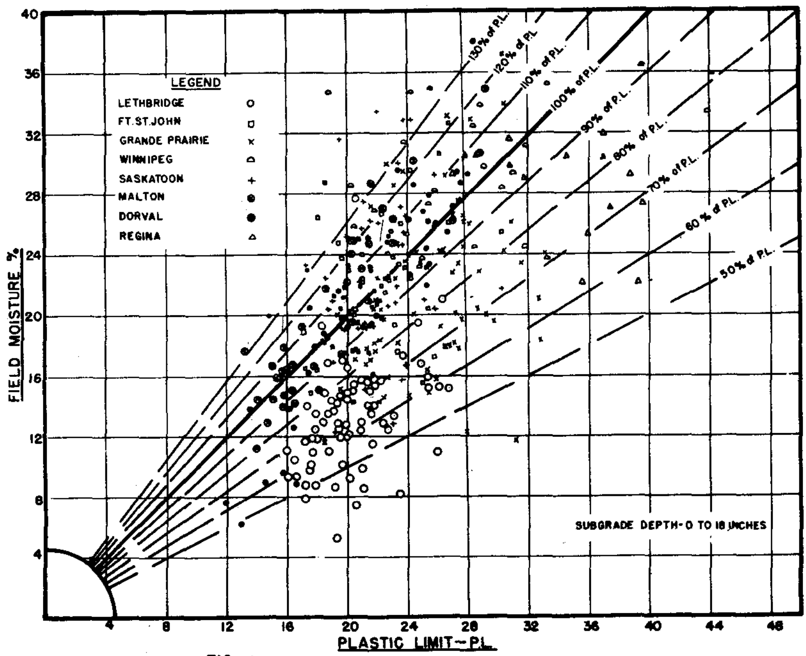


FIG. 4 FIELD MOISTURE VERSUS PLASTIC LIMIT—P.L.

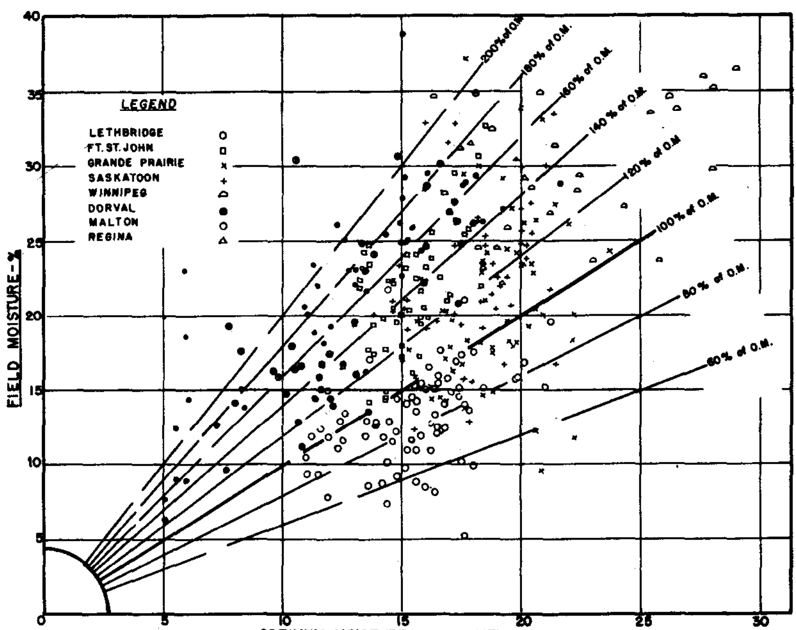


FIG. 5 FIELD MOISTURE AS % VERSUS OPTIMUM MOISTURE AS % (MODIFIED AASHO)

It is evident from Fig. 3 that complete saturation of the subgrade occurred at a relatively small percentage of the total number of subgrade locations tested. Even if all values above 90 per cent saturation were considered to represent complete saturation, the percentage of locations that could be considered to be saturated is only 21.7 per cent of the total.

Kersten[5] in summarizing a study of moisture contents in highway subgrades, reports that for clay soils the field moisture content generally exceeds the plastic limit. It is interesting to note that the reverse has been the case for the eight airports with clay subgrades included in this study.

Fig. 5 indicates that the field moisture content of the clay subgrades at the eight airports exceeded the modified A.A.S.H.O. optimum in 71.2 per cent of the locations tested. Figs. 4 and 5 can also be usefully employed when estimating the probable subgrade moisture content to be expected under paved runways at a new site.

5. Plate Bearing Tests.

(a) Equipment

To determine the supporting capacity of the subgrades, base courses, and surfaces of the existing runways, repetitive loading with steel bearing plates was employed. The arrangement of the load testing equipment followed in general that recommended by the Committee on Flexible Pavement Design of the Highwey Research Board[3].

Four weighted tractor trailer units similar to that of Fig. 6, and capable of applying loads of from 70,000 to over 100,000 pounds were employed as the source of reaction.

The arrangement of the equipment for performing each load test is illustrated in Fig. 7. Circular steel plates 1 inch thick and 30 inches in diameter were used for most tests, but a considerable number were performed with bearing plates 12, 18, 24, 36 and 42 inches in diameter. Measured load was transferred from a jacking point on the trailer to the steel bearing plate by means of hydraulic jacks of 100,000 pounds capacity. Deflections of the bearing plate were measured to the nearest 1/10,000 of an inch by means of two Ames dials graduated in increments of 1/1000 of an inch, set on the plate near the extremities of a diameter, Fig. 7.

All points of support for either the tractor-trailer units, or for the deflection beam were at least 8 feet from the bearing plate.

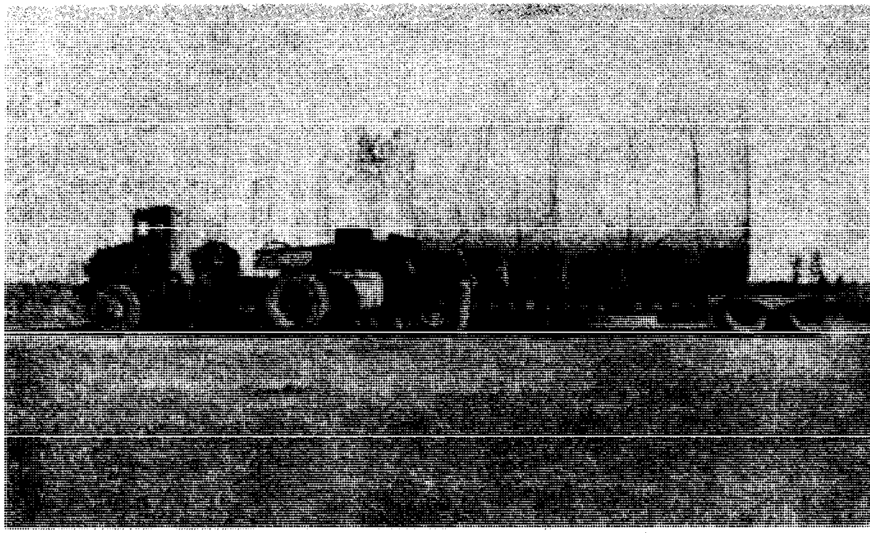


Fig. 6a. Load test unit No. 1. Capacity 150,000 pounds.

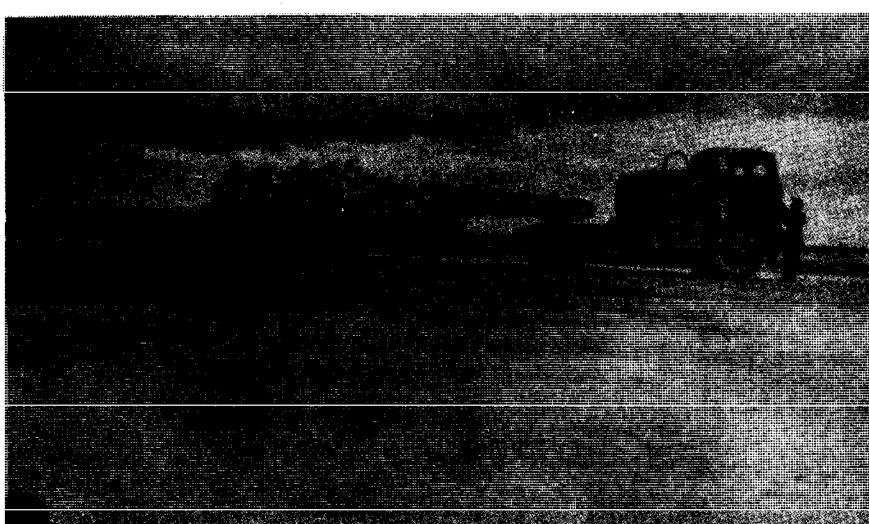


Fig. 6b. Load test unit No. 2. Capacity 80,000 pounds.

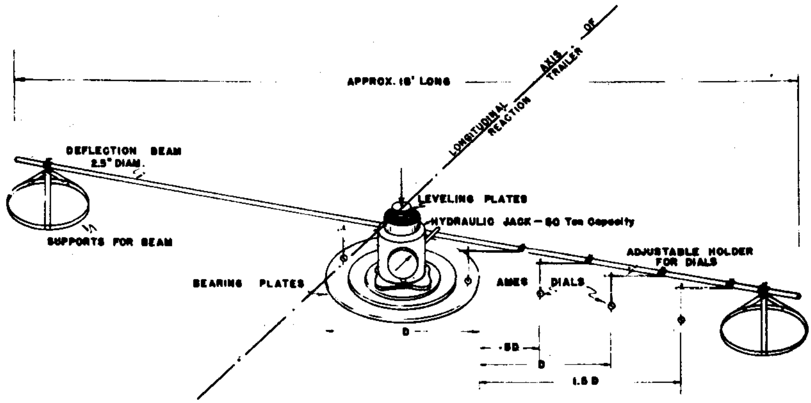


DIAGRAM SHOWING ARRANGEMENT OF EQUIPMENT FOR BEARING TEST

FIG. 7

(b) Load Test Procedure

The load test procedure employed, while following in general that recommended by the Highway Research Board Committee on Flexible Pavement Design[3], was also governed by the need for obtaining test data on which the design of either rigid or flexible pavements could be based, if it should become necessary to reconstruct or extend the runways at any one or more of the airports investigated.

It was therefore necessary to employ one loading of a magnitude which would give a deflection of approximately 0.05 inch, from which the subgrade modulus for rigid pavement design could be determined. Another load, giving a deflection of about 0.5 inch was required to provide data for flexible pavement design. A third load intermediate between these was used, to give the further information required for a complete load deflection curve.

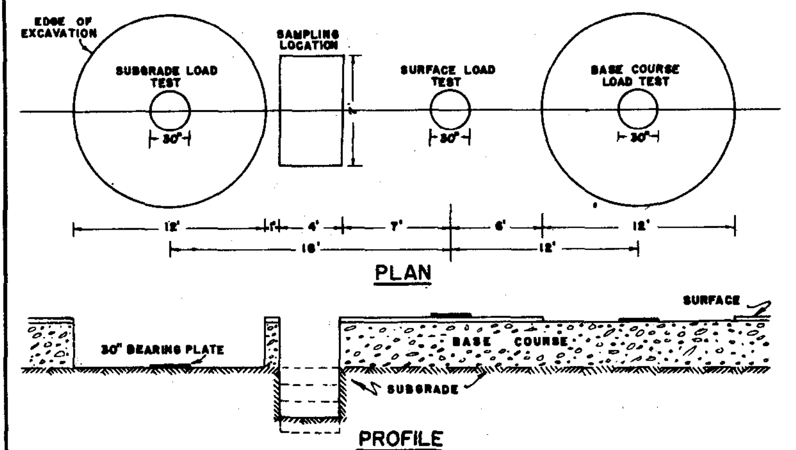
Each load was applied and released from four to six times. The end point for either application or release of each repetition of load was a rate of deflection of 0.001 inch or less per minute for each of three successive minutes.

Load tests were made on the surface of the pavement, on the surface of the base course, and on the surface of the subgrade. Fig. 8 illustrates the general arrangement for the grouping of the load tests at each test location on a runway, followed in 1945. For load tests on the base course, the pavement was removed from a circular area 12 feet in diameter, and the bearing plate was placed in the centre. For load tests on the subgrade, both pavement and base course were excavated to the top of the subgrade over a circular area 12 feet in diameter, in order that the subgrade load test would be completely unconfined.

All field tests were performed, and both disturbed and undisturbed samples were obtained, in the rectangular sampling area situated between the surface and subgrade load tests, Fig. 8.

FIG. 8

DIAGRAM SHOWING TYPICAL ARRANGEMENT OF LOAD TESTS & SAMPLE LOCATIONS



Since the Department of Transport has no large central laboratory of its own, arrangements were made to have the required tests on the disturbed and undisturbed soil samples performed at the engineering laboratories of the University of Toronto, McGill University, and the University of Alberta.

It might be added that for a time the investigation required more than one hundred employees for the various phases of work involved in the field and laboratory testing.

(c) <u>Plotting of Load Test Data for Load Deflection Curves.</u>

To obtain the data needed for the construction of load versus deflection curves, the following steps were involved:

(1) For each repetition of each load, the deflection was determined at which the rate of deflection was exactly 0.001 inch per minute. This can be found with sufficient accuracy from inspection of the deflection data for each repetition of load recorded in the field note books.

(2) Zero point corrections are determined for both applied load and deflection. This requires taking into account the weight of the jack, the pyramid of bearing plates, etc., and the corrected jack loads at which the deflection gauges are zeroed at the beginning of the test. The load correction may amount to from one to three thousand pounds.

The zero point correction for deflection is obtained graphically, and occasionally may amount to two or three hundredths of an inch. It must be added algebraically to the observed deflections.

(3) The corrected deflections (at which the rate of deflection is exactly 0.001 inch per minute for each repetition of each load), versus the logarithm of the number of repetitions of load is plotted for the three corrected loads on semi-log paper, Fig. 9.

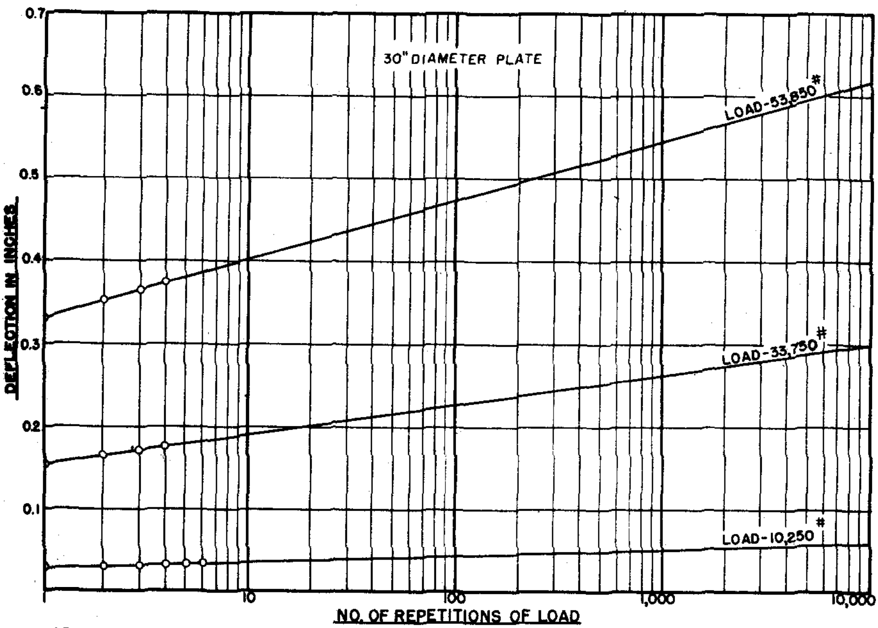


FIG. 9 INFLUENCE ON DEFLECTION OF REPETITIONS OF LOADING FOR EACH OF THREE MAGNITUDES OF LOAD

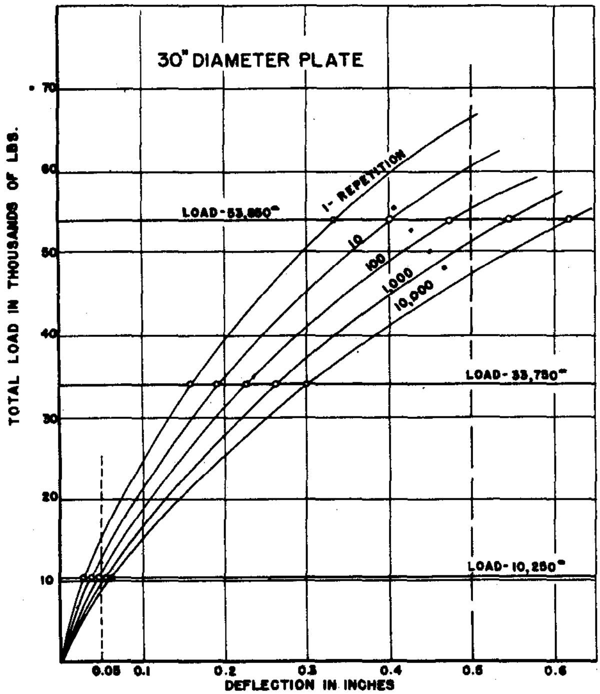


FIG.10—LOAD VERSUS DEFLECTION FOR REPEATED LOADINGS

(4) Fig. 10 was prepared directly from Fig. 9. The curves from top to bottom represent load versus deflection for 1, 10, 100, 1000 and 10,000 repetitions of load, respectively.
From Fig. 10, data for either rigid or flexible pavement design can be obtained for any number of repetitions of load. The subgrade modulus "k" for rigid pavement design, can be calculated from the load for 0.05 inch deflection, while for flexible pavement design, the load corresponding to 0.5 inch deflection can be used.

6. Runway Load Test Data Versus Safe Design for Aeroplane Wheel Loadings.

The maximum wheel loadings which the runways have been supporting under reasonably intensive data can be estimated from the traffic data. This estimate is somewhat complicated by the fact that the runways also serve as taxiways at most Canadian airports. Experience has shown that a greater thickness of base and surface is required for taxiways aprons, and turnarounds, than for runways, for the same aeroplane wheel loading.

Load test data for any runway may vary by several thousand pounds between the high and low values. To avoid either serious over-design or under-design, the load test value at the lower 25 per cent point (the *lower quartile* point) was adopted as the representative value for each runway. That is, the representative load test value was greater than 25 per cent, but smaller than 75 per cent of the load test results obtained.

It was found that the lower quartile plate bearing value (the lower 25 per cent point) at 0.5 inch deflection for 10 repetitions of load, provided a load test value that appeared to be approximately equal to the maximum wheel load which the runways had been supporting under reasonably intensive traffic. It should be emphasized that both wheel load and representative plate bearing value must apply to the same contact area.

Further information may indicate that this approach should be modified, but since it seems to fit in with present traffic experience in Canada, it is employed as the criterion for safe runway design throughout this paper. It is for this reason that the data for most of the accompanying diagrams are for 10 repetitions of load.

7. <u>General Information from Load Test Data</u>.

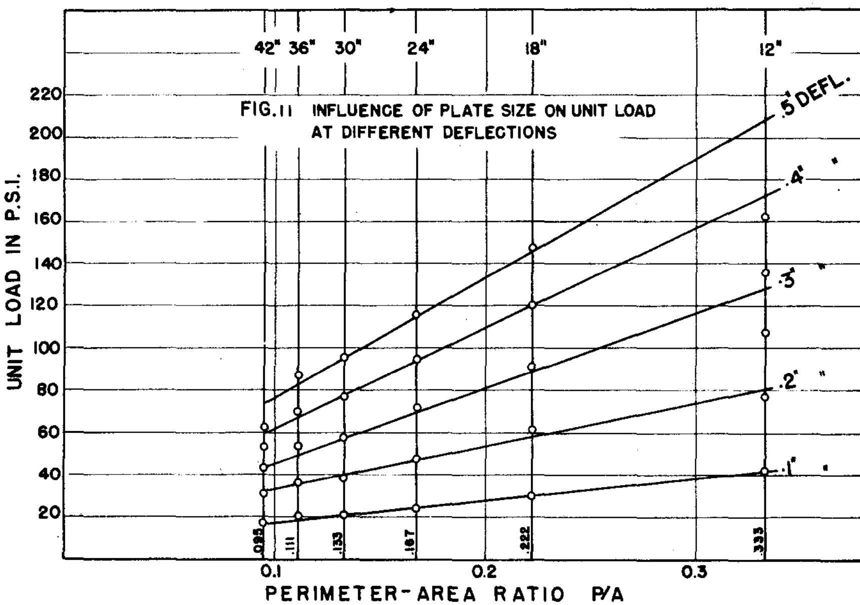
(a) <u>Influence of P/A Ratio on Unit Load Bearing Capacity</u>.

It has been known for many years from the work of early investigators in soil mechanics, and more recently from the investigations of Housel[7], Hubbard and Field[8], Campen and Smith [9,10], Teller and Sutherland[11], Middlebrooks and Bentram[12], and others, that the size of bearing plate employed for load tests on soils, materially influences the magnitude of the unit load which is supported at a given deflection. For cohesive soils, the influence of plate size on unit load is frequently expressed as a straight line graph when unit load is plotted versus the perimeter area (P/A) ratio of bearing plates of different diameters.

It has been suggested recently[13], that the size of the bearing plate ceases to have any influence on the magnitude of the unit load supported at a given deflection, if the plate diameter is greater than about 26 to 30 inches. Professor Housel's investigations on the other hand, have indicated that the straight line graph of unit load versus P/A ratio holds for bearing plates up to at least 40 inches in diameter, and probably well beyond.

To obtain further information on this matter, the Department of Transport made a considerable number of tests with bearing plates 12, 18, 24, 30, 36 and 42 inches in diameter. When the values of unit load are plotted versus the P/A ratio for these different plates at any given deflection, graphs similar to that shown in Fig. 11 are obtained.

FIG. 11 INFLUENCE OF PLATE SIZE ON UNIT LOAD AT DIFFERENT DEFLECTIONS



When all of the load tests with bearing plates of different sizes are considered, there seems to be little doubt that a straight line relationship exists between unit load support at a given deflection versus P/A ratio, for bearing plates with diameters between 12 and 42 inches, and probably larger.

These results, therefore, confirm those of Housel on the influence of bearing plate size on unit subgrade support, and are indicated also by the investigations of Campen and Smith[9,10]

(b) <u>Ratios of Loads Supported on Given Bearing Plate at Different Numbers of Repetitions</u>.

Fig. 12 indicates that a ratio appears to exist between the load carried at 1 repetition of load to that carried at 10 repetitions of load, for a 30-inch diameter bearing plate at 0.5 inch deflection. Similar ratios seem to

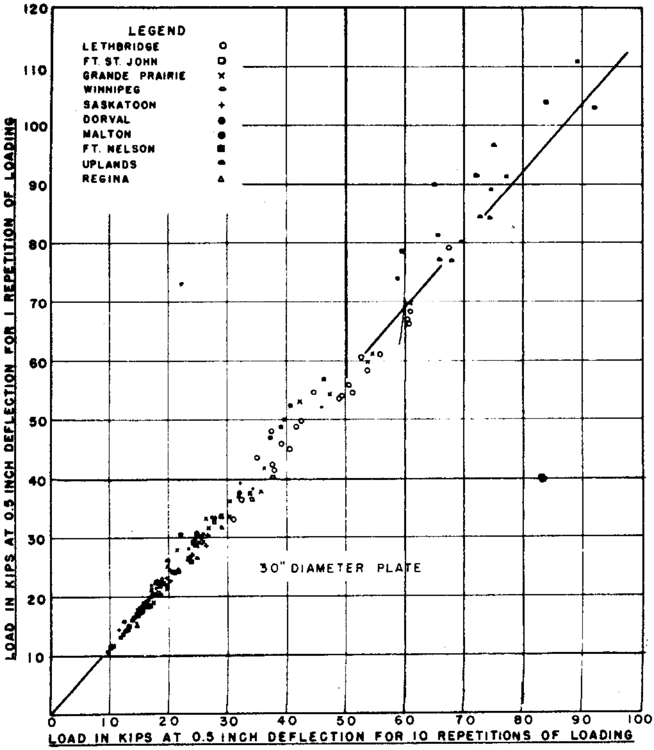


FIG. 12 LOAD IN KIPS AT 0.5 INCH DEFLECTION FOR 1 REPETITION VERSUS LOAD IN KIPS AT 0.5 INCH DEFLECTION FOR 10 REPETITIONS

hold for 10 versus 100 repetitions, and 10 versus 1000 repetitions, over the range from 0.0 to 0.7 inch deflection. These ratios are summarized in Table 4, for deflections between 0.2 to 0.7 inch.

Table 4

Ratio $\dfrac{\text{Load for "n" repetitions at deflection indicated}}{\text{Load for 10 repetitions at same deflection}}$

Deflection Range Inches	Number of Repetitions of Load 1	10	100	1000
0.2 to 0.7	1.15	1.00	0.89	0.80

The ratios of Table 4 are convenient when designing for more limited or for heavier traffic, than the load indicated for 0.5 inch deflection at 10 repetitions of load, which is employed in this paper as a criterion for safe runway design.

(c) Ratios of Loads Supported on a Given Bearing Plate at Different Deflections.

When the curves for load versus deflection for load tests with the 30-inch diameter plate on the subgrades at all ten airports were analyzed, the relationships illustrated in Figs. 13 and 14 were developed. This information is summarized in Fig. 15, as an arithmetic graph of the ratio of load supported at any deflection up to 0.7 inch over load carried at 0.2 inch deflection, versus deflection in inches.

Fig. 15 indicates that if the exact load supported at 0.2 *inch deflection can be accurately determined* for a 30-inch diameter plate, the complete load deflection curve can be calculated over the range of deflection between 0.0 and 0.7 inch.

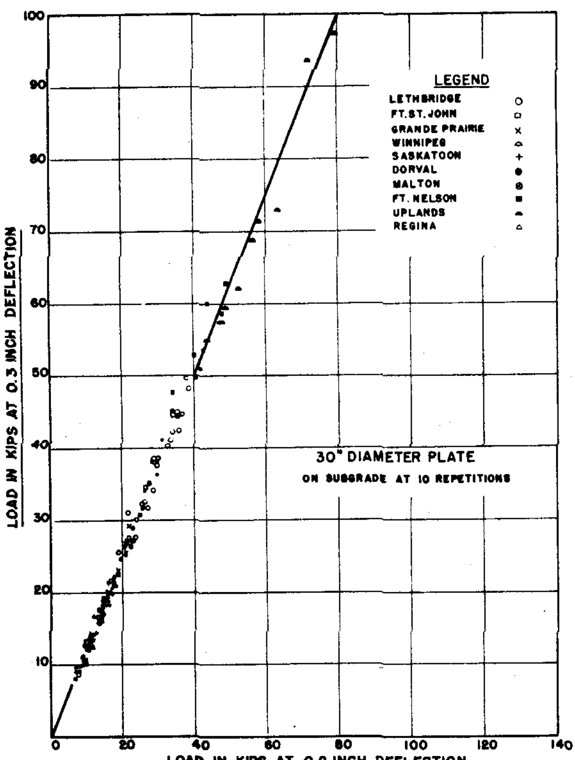


FIG. 13 LOAD IN KIPS AT 0.3 INCH DEFLECTION VERSUS LOAD IN KIPS AT 0.2 INCH DEFLECTION

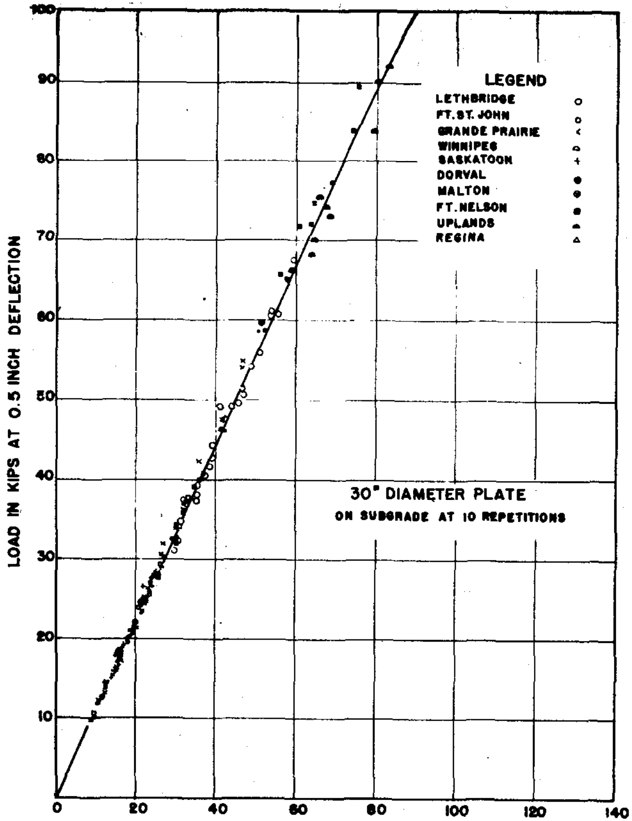


FIG. 14 LOAD IN KIPS AT 0.5 INCH DEFLECTION VERSUS LOAD IN KIPS AT 0.4 INCH DEFLECTION

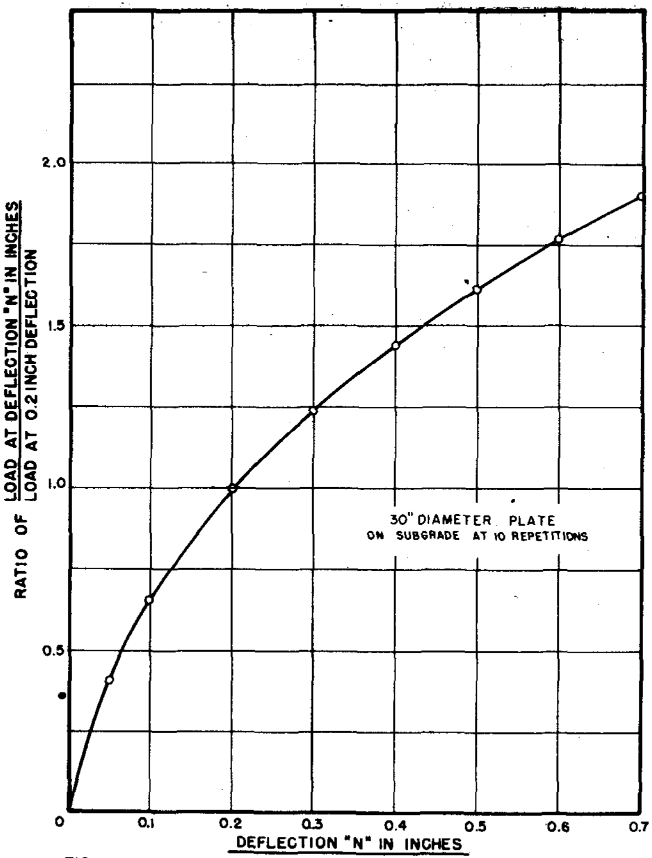


FIG. 15 RATIO OF LOAD AT DEFLECTION "N" IN INCHES OVER LOAD AT 0.2 INCH DEFLECTION VERSUS DEFLECTION "N" IN INCHES.

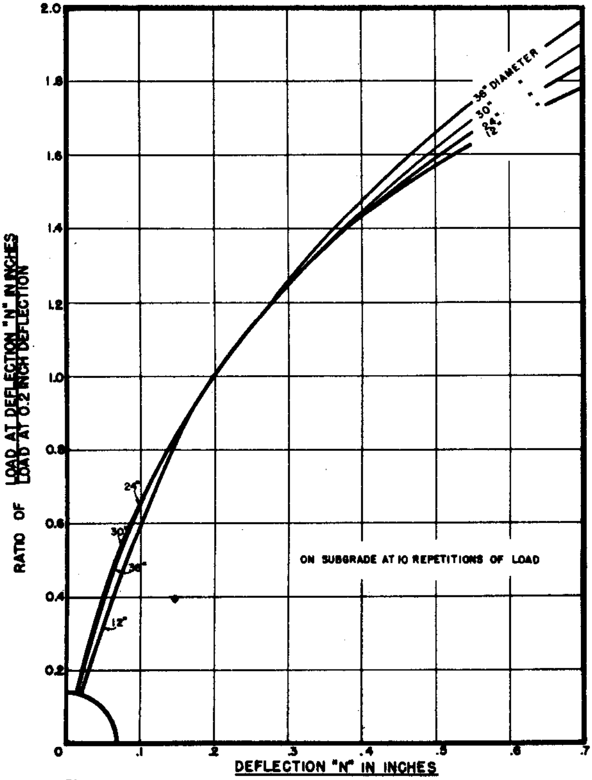


FIG. 16 RATIO OF LOAD AT DEFLECTION "N" IN INCHES OVER LOAD AT 0.2 INCH DEFLECTION VERSUS DEFLECTION "N" IN INCHES FOR BEARING PLATES 36, 30, 24, AND 12 INCHES IN DIAMETER.

Information similar to that in Figs. 13 and 14 has been assembled for 36-inch, 24-inch and 12-inch diameter plates with very similar results, Fig. 16. It may be seen that the ratios for bearing plate diameters of 36, 24, and 12 inches do not coincide with those for the 30-inch plate.

Similar information which has been developed for load tests made on the surfaces of flexible pavements, for bearing plate diameters of 12, 24, 30 and 36 inches, and for a deflection range of 0.0 to 0.7 inches, is summarized in Fig. 17.

(d) Ratios of Loads Supported on Bearing Plates of Different Sizes at Same Deflection.

In Fig. 18 the total load carried on a 36-inch plate is plotted versus the total load supported by a 30-inch plate at a deflection of 0.2 inch. A straight line relationship seems to be indicated.

Information similar to that of Fig. 18 was developed at deflections of 0.05, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6 and 0.7 inches, for 12-inch versus 30-inch plates, 18-inch versus 30-inch plates, 24-inch versus 30-inch plates, 36-inch versus 30-inch plates and 42-inch versus 30-inch plates. From this information, the ratio of the unit load supported

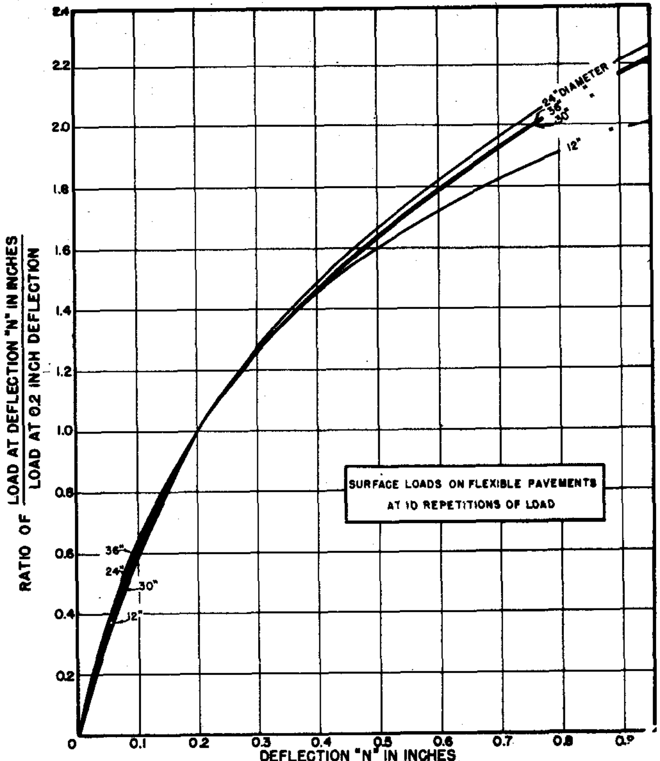


FIG. 17 RATIO OF LOAD AT DEFLECTION "N" IN INCHES OVER LOAD AT 0.2 INCH DEFLECTION VERSUS DEFLECTION "N" IN INCHES FOR BEARING PLATES OF 36, 30, 24, AND 12 INCHES IN DIAMETER.

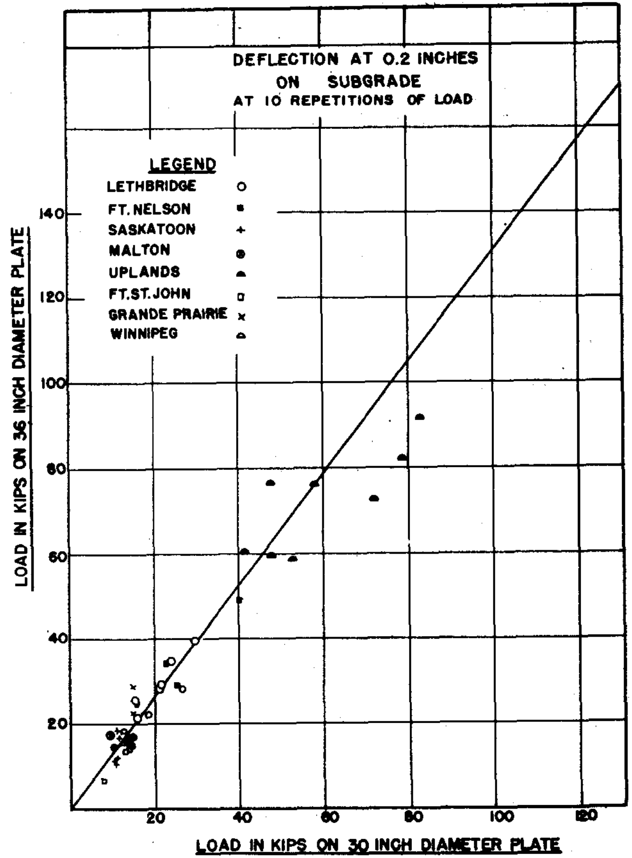


FIG. 18 LOAD IN KIPS ON 36" DIAMETER PLATE VERSUS LOAD IN KIPS ON 30" DIAMETER PLATE AT 0.2 INCH DEFLECTION

on a plate of given size over the unit load supported on a 30-inch diameter plate could be readily determined for any required deflection.

Knowing the ratio of the load supported at one deflection to that supported at another deflection for a bearing plate of given size, e.g., Fig. 15 for the 30-inch plate, and knowing also the ratio of the unit load supported on a plate of one size to the unit load supported on a plate of different size for a given deflection, it is a relatively simple matter to prepare the chart of Fig. 19. This chart is based upon the load carried by a 30-inch diameter plate at 0.2 inch deflection as unity, or it could be considered to be based upon a unit load of 1 p.s.i. on a 30-inch diameter plate at 0.2 inch deflection.

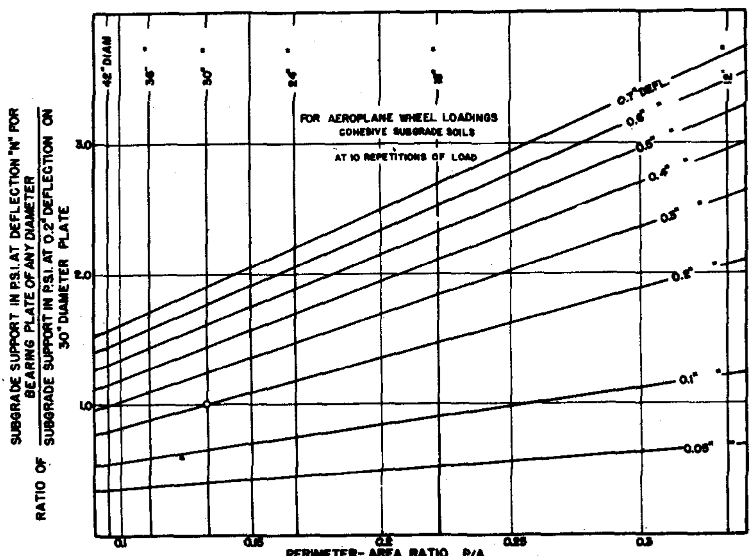


FIG.19 RATIO OF SUBGRADE SUPPORT IN P.S.I. AT DEFLECTION "N" FOR BEARING PLATE OF ANY DIAMETER OVER SUBGRADE SUPPORT IN P.S.I. AT 0.2" DEFLECTION ON 30" DIAMETER PLATE VERSUS PERIMETER AREA RATIO.

The value of Fig. 19 lies in the fact that if the unit load supported on a 30-inch diameter plate at 0.2 inch deflection is known accurately, the unit load supported on a bearing plate for any other diameter over the range of 12 to 42 inches and probably somewhat beyond, can be calculated for any deflection between 0.0 and 0.7 inch.

From load tests made on the bituminous surfaces at the ten airports, information similar to that of Fig. 19 for subgrades, was obtained for flexible pavements, Fig. 20.

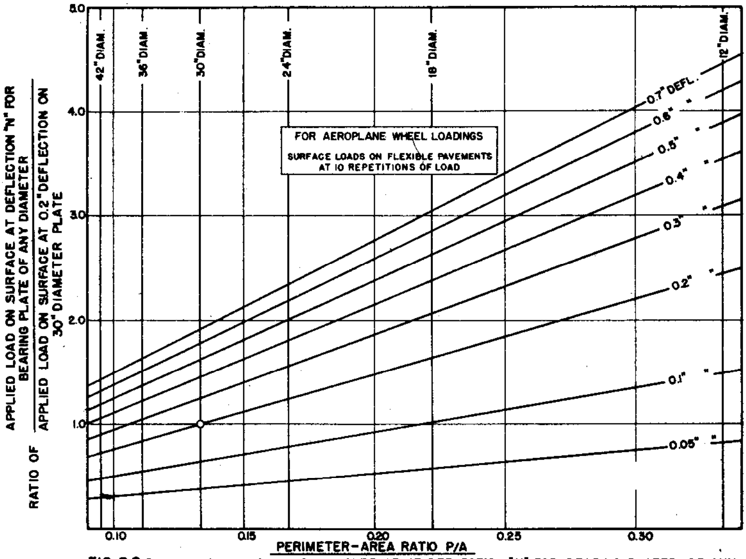


FIG. 20 RATIO OF APPLIED LOAD ON SURFACE AT DEFLECTION "N" FOR BEARING PLATES OF ANY DIAMETER OVER APPLIED LOAD ON SURFACE AT 0.2" DEFLECTION ON 30" DIAMETER PLATE VERSUS PERIMETER AREA RATIO.

8. <u>Yield Point Deflections for Subgrades and Flexible Pavements</u>.

Professor Housel has devised a method for determining the yield point of a soil, when load tests have been performed with bearing plates of at least three different sizes[7,14]. Professor Housel defines the yield point, or bearing capacity limit of a soil, as the maximum load which a soil will support without progressive settlement occurring [15,16]. For loads beyond the bearing capacity limit, deflection increases progressively with time. The yield point deflection is the deflection which occurs under the yield point load, or bearing capacity limit of the soil.

Professor Housel's method would require too much space to outline here, other than to state that it depends upon perimeter shear "m", developed pressure "n", and deflection "d" under each magnitude of load. From data for each of these variables, he calculates soil resistance coefficients K_1 and K_2, where $K_1 = d/n$, and $K_2 = m/n$. When K_1 and K_2 values are plotted versus deflection, the yield point deflection occurs at either a minimum value of the K_1 curve, or at a maximum value of the K_2 curve.

The diagrams of Figs. 19 and 20 are susceptible to analysis by Housel's method, and the results are presented in Figs. 21 and 22, respectively.

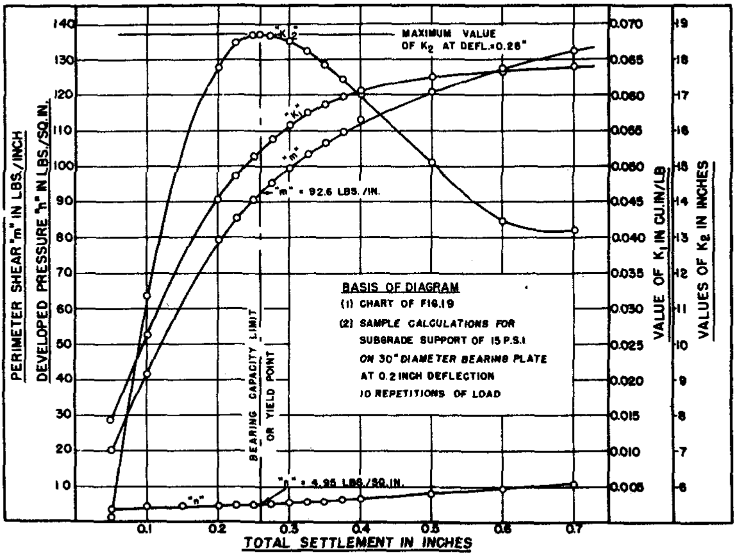


FIG.21 YIELD POINT DIAGRAM FOR COHESIVE SUBGRADES

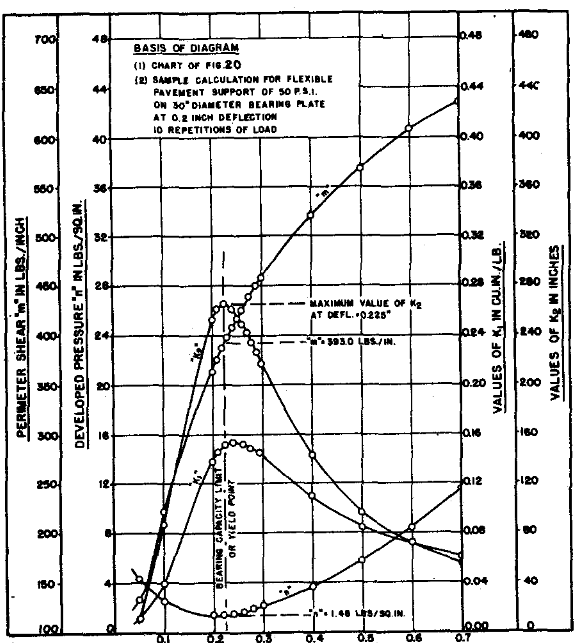


FIG.22 YIELD POINT DIAGRAM FOR FLEXIBLE PAVEMENTS

Fig. 21 indicates that the yield point occurs at a deflection of 0.26 inch, where the K_2 curve reaches its maximum. This means that the average yield point deflection of the subgrades for the ten airports is 0.26 inch.

Fig. 22 gives the average yield point deflection for the flexible pavements at the ten airports. It also occurs at a maximum value of K_2, and has the value of 0.225 inch.

The very nearly identical yield point deflections indicated by Figs. 21 and 22 respectively, could be interpreted as evidence that the subgrade is the weakest element of the runway structure, and that it is the yield point of subgrade which established the yield point of the flexible pavements at these ten airports.

9. Base Course Load Support Versus Nature of Base Course Materials.

Table 2 indicates that the base courses at the various airports tested so far, consist of several materials, including pit-run and crusher-run gravel, waterbound macadam, and mechanical stabilization.

At a considerable number of test locations for the different airports investigated, load tests were performed on the subgrade, base course, and wearing surface. Load versus deflection curves were prepared for the load tests on each of these three elements of the runway structure at each test location, e.g., Fig. 23. Apart from the data for one airport, the analysis of these load deflection curves has in general provided no definite evidence that any one of these types of granular base course materials is superior to any other type, insofar as load supporting value per inch of thickness is concerned. The U.S.E.D. appear to have obtained data which point to a similar conclusion[6,17]

10. Bituminous Surface Stronger than Granular Base.

A study of the load deflection curves for subgrade, base course and surface, Fig. 23, at the different test locations, showed very definitely that the load carrying capacity of a bituminous surface was greater than that of the various types of base course per inch of thickness. The test data indicate that for bituminous surfaces made with liquid asphalts, soft asphalt cements (softer than about 120 penetration), etc., one inch of thickness has the same load supporting capacity as about 1.5 inches of granular base. This ratio should probably not be applied for greater thicknesses of these types of pavement than about 4 inches, until justified or otherwise for greater thicknesses by further load tests.

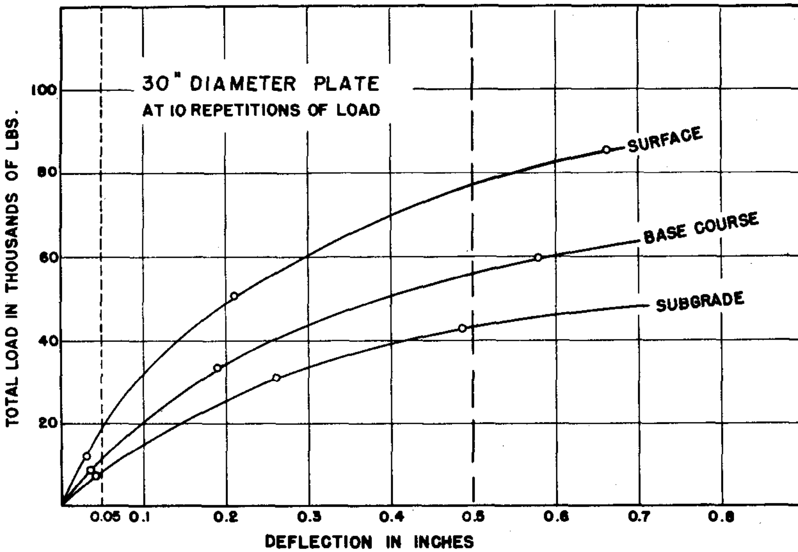


FIG.23 LOAD VERSUS DEFLECTION CURVES (Normal) FOR SUBGRADE, BASE COURSE AND SURFACE

For well designed and constructed bituminous concrete, penetration macadam, and sheet asphalt, one inch of thickness of these types appears to have the same load carrying capacity as about 2.5 inches of granular base. Again, however, this ratio should probably not be applied for a greater thickness of these types of pavement than about 6 inches, unless warranted by further investigation.

It is to be noted that this information with regard to the relative load carrying capacities of bituminous pavements versus granular bases was obtained on runways that had been in service for several years. There has not yet been an opportunity to learn whether the same ratios would hold for either newly constructed or relatively new bituminous surfaces.

11. Subgrade Load Test Versus California Bearing Ratio.

One of the objectives of the investigation was to establish relationships between subgrade load tests and certain simple field tests such as the California Bearing Ratio (C.B.R.), cone bearing, and Housel penetrometer. If these relationships could be established, subgrade bearing capacity data could be obtained by means of one or more of these simple field tests in place of the cumbersome and costly load test.

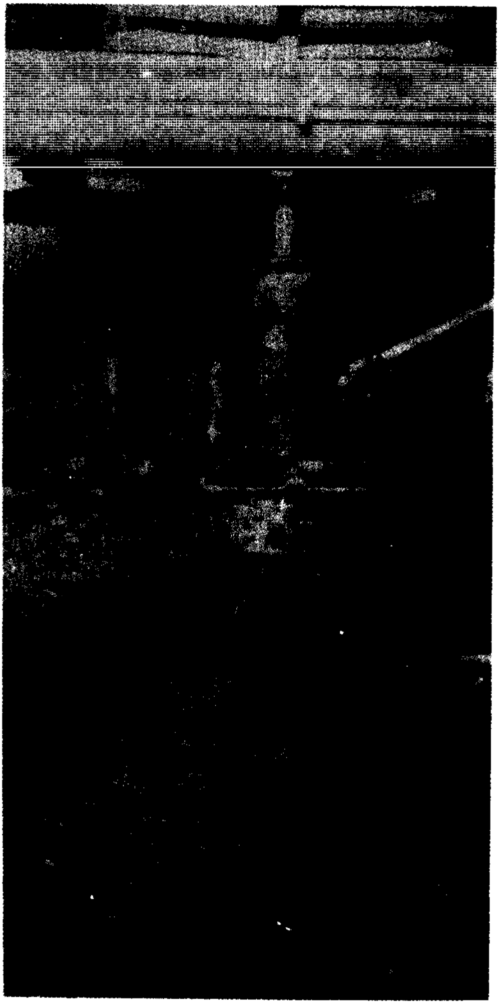


Fig. 24. Equipment for obtaining C.B.R. Samples.

At all test locations, undisturbed samples were taken in cylinders 6 inches in diameter by 6 inches high, Fig. 24, at depths of 0 to 6 inches and 9 to 15 inches below the surface of the subgrade. They were immediately trimmed and sealed and then shipped to the laboratory for the determination of C.B.R. values for both field and soaked conditions.

The field C.B.R. values were obtained by testing in the "as received" condition. The soaked C.B.R. values were determined after soaking the samples for four days according to standard procedure, and with required surcharge.

For the eight airports where the subgrade consists of cohesive soil, Table 5 lists the C.B.R. values obtained for both field and soaked conditions.

Table 5

C.B.R. values for both field and soaked conditions, for eight airports where the subgrade consists of cohesive soil.

	C.B.R. Values					
	Field			Soaked		
Airfield	Ave.	Max.	Min.	Ave.	Max.	Min.
Fort St. John	5.1	15.8	2.3	2.8	4.7	1.1
Grande Prairie	6.1	14.5	2.4	2.2	3.9	1.0
Saskatoon	5.3	10.4	2.0	3.6	6.8	1.5
Lethbridge	12.6	25.0	5.4	4.6	7.3	1.8
Dorval	3.9	4.9	2.7	3.1	3.9	2.2
Winnipeg	4.7	6.8	3.4	3.3	5.3	2.6
Malton	6.6	13.5	2.9	3.5	4.8	2.2
Regina	7.3	11.2	5.7	3.3	5.0	1.2

The average soaked C.B.R. rating of the subgrades varied from approximately 2.3 to 4.5 for the eight airports.

The field C.B.R. values, on the other hand, varied with the actual condition of the subgrade soil under the pavement, and the average ratings ranged from 3.9 to 12.6.

In Fig. 25, the field C.B.R. values are plotted versus actual subgrade support measured for a 30-inch diameter plate at 0.2 inch deflection, for the various test locations at the eight airports with cohesive subgrade soils. The field C.B.R. value employed in each case was the average for subgrade depths of 0 to 6 and 9 to 15 inches.

In arriving at the location of the best average line through the points of Fig. 25, the data could have been treated statistically without regard for the different airports to which they pertained. On this basis, however, the field C.B.R. values could lead to seriously overestimating the actual subgrade support for some of the airports. For this reason, a different approach was adopted. The position of the best average line through the data of Fig. 25 was established on the basis that the subgrade support indicated by the curve for field C.B.R. values must not exceed the actual subgrade support determined by a 30-inch diameter

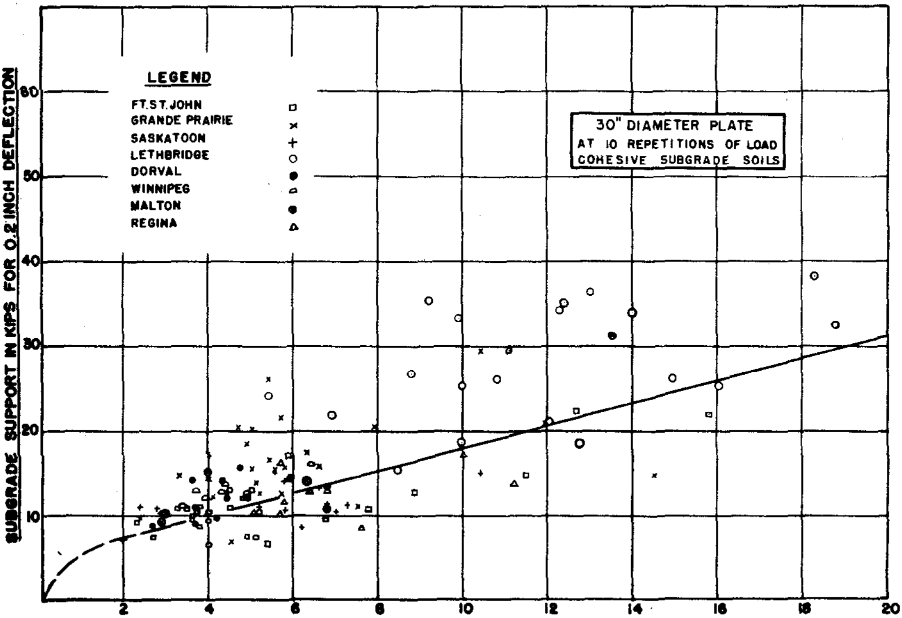


FIG. 25 SUBGRADE SUPPORT VERSUS FIELD C.B.R.

plate at 0.2 inch deflection, by more than about 10 per cent for any one of the eight airports. That is, the location of the best average line through the data must not lead to overestimating the true subgrade support by more than about 10 per cent when the field C.B.R. test is employed to measure subgrade bearing capacity indirectly.

12. Subgrade Load Test Versus Cone Bearing.

Boyd[18], has described a cone bearing test that can be performed rapidly with very simple equipment, Fig. 26, with which he evaluated subgrades for flexible pavement design for highways in North Dakota. The cone bearing test is made by loading a standard steel cone with 10, 20, 40 and 80 pounds, in turn, and reading the penetration of the cone into the subgrade after each load in succession has been applied for one minute. From 4 to 6 determinations should be made on the surface of each layer of subgrade tested in order to obtain good average values. Good checks can be obtained and all values that deviate too widely from the average should be discarded.

Fig. 26. Equipment for Cone Bearing test.

In Fig. 27 cone bearing values are plotted versus subgrade support on a 30-inch diameter plate at 0.2 inch deflection for test locations at the eight airports with cohesive subgrade soils. The cone bearing value employed in each case was the average for the three subgrade layers, 0 to 6, 6 to 12 and 12 to 18 inches below the top of the

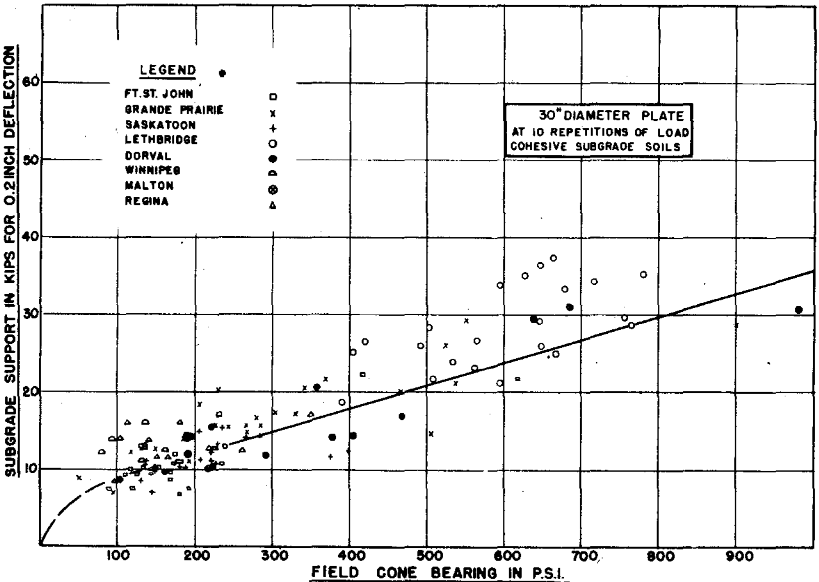


FIG.27 SUBGRADE SUPPORT VERSUS FIELD CONE BEARING

subgrade, with the exception that where the value for the 0 to 6 inch layer was higher than for the other two, it was discarded and the average was based upon values for the 6 to 12 and 12 to 18 inch layers. This gave better agreement with the load test data.

The position of the best average line through the data of Fig. 27, was established on the basis that the subgrade support indicated by the cone bearing values curve, would not exceed the true subgrade support measured by a 30-inch diameter plate at 0.2 inch deflection by more than about 10 per cent for any one of the eight airports included.

13. Subgrade Load Test Versus Housel Penetrometer.

Professor Housel has investigated the possible correlation of a simple penetrometer test with his own load test data[14]. The penetrometer test equipment[19], Fig. 28, consists of a sharpened 1-1/4 inch diameter standard pipe, which with accessories weighs exactly 20 pounds, exclusive of the driving weight, which also weighs exactly 20 pounds. Stops on the barrel of the 1-1/4 inch pipe control the height of drop of the driving weight to exactly 34 inches. The test consists of determining the number of blows of the 20 pound driving weight falling exactly 34 inches required to drive the sharpened pipe 6 inches into the soil. A cardboard strip

Fig. 28. Equipment for Housel penetrometer test.

firmly attached to the barrel of the pipe, Fig. 28, is marked with pencil at the beginning of the test and after the penetration of each blow.

Fig. 29 is a graph of Housel penetrometer values versus subgrade support on a 30-inch diameter plate at 0.2 inch deflection, for test locations at the eight airports with cohesive subgrade soils. The penetrometer value taken in each case was the average for three subgrade layers 0 to 6, 6 to 12 and 12 to 18 inches below the surface of the subgrade.

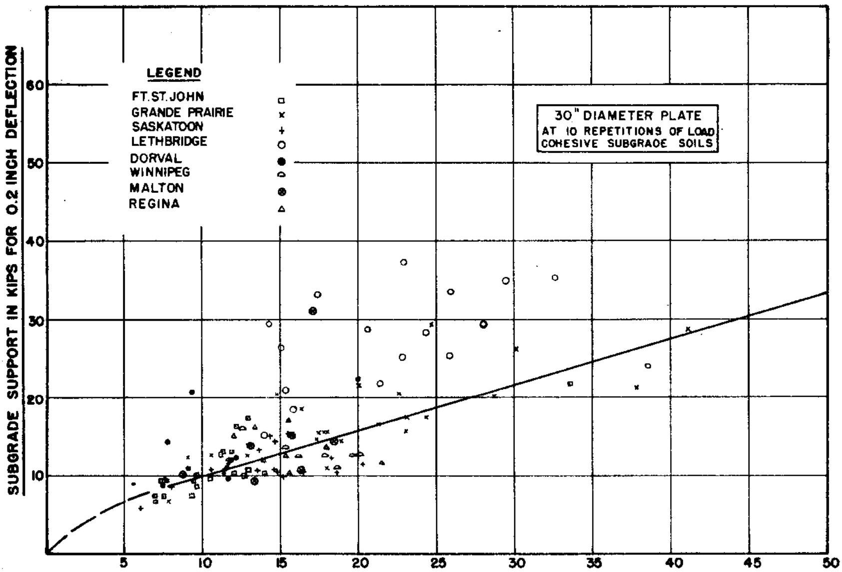


FIG. 29 SUBGRADE SUPPORT VERSUS HOUSEL PENETROMETER

14. Subgrade Load Test Versus Triaxial Compression Test.

In addition to field C.B.R., cone bearing, and Housel penetrometer tests, a laboratory test was desired, which could be correlated with subgrade bearing values determined from load tests on steel bearing plates. The triaxial compression test was selected for this purpose, and a relatively large amount of triaxial compression test data was obtained on undisturbed samples sent in from the field.

The data obtained from a triaxial compression test is usually plotted as a Mohr diagram, which provides information concerning the quantities, cohesion "c," angle of internal friction φ, and various corresponding equilibrium values of vertical pressure V and lateral pressure L.

If the triaxial compression test was to be correlated with the plate bearing test, some characteristic value from the triaxial test was required, which would be as representative and as quantitatively definite as the cone bearing or C.B.R. values, for example, are for the cone bearing or C.B.R. tests, respectively. From the Mohr diagram, it is obvious that this representative value for the triaxial compression test must come from the cohesion "c," the angle of internal friction "φ," the lateral pressure "L," the vertical

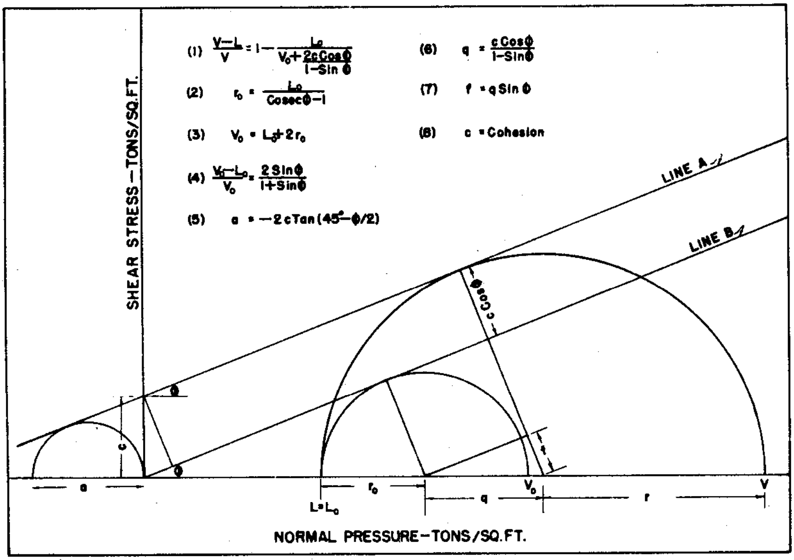


FIG.30 DIAGRAM ILLUSTRATING CERTAIN GEOMETRICAL RELATIONSHIPS FOR TRIAXIAL COMPRESSION TEST DATA

pressure "V," or from some combination of two or more of these variables.

Fig. 30 illustrates the geometrical and trigonometrical relationships that are employed for the development which follows in this section. The two Mohr rupture lines in Fig. 30, "line A" and "line B," are parallel, but the cohesion "c" is zero for line B which passes through the origin. The diagram has been constructed in such manner that the two Mohr circles have the same value for lateral pressure, that is "L" = "L_o." However, the corresponding vertical pressure "V" for line A is greater than the vertical pressure "V_o" for line B.

The equations required for the following paragraphs are listed in Fig. 30.

When studying the relationships between the different variables of the Mohr diagram, it was found that a rectangular hyperbola resulted if (V-L)/V was plotted versus L, Fig. 31. This curve conformed to the general equation

$$(x - a)\ (y - b) = K$$

where each symbol has the significance indicated in Fig. 31, and K is a constant.

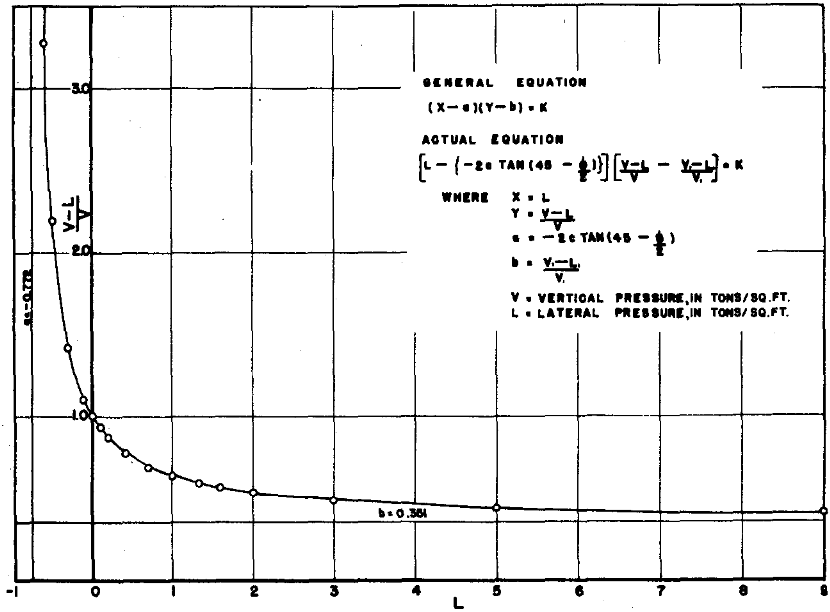


FIG. 31 RATIO OF $\frac{V-L}{V}$ VERSUS LATERAL PRESSURE L FOR TRIAXIAL COMPRESSION TEST

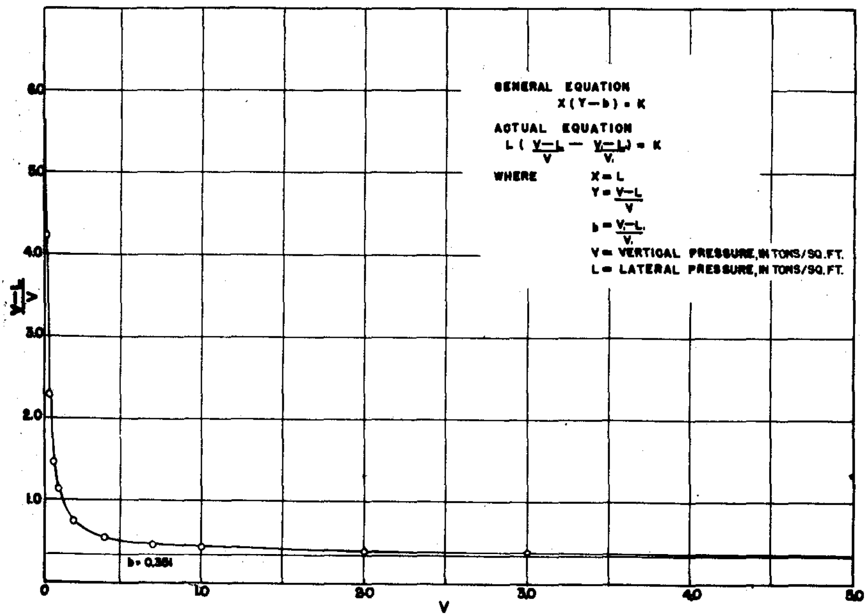


FIG. 32 RATIO OF $\frac{V-L}{V}$ VERSUS VERTICAL PRESSURE V FOR TRIAXIAL COMPRESSION TEST

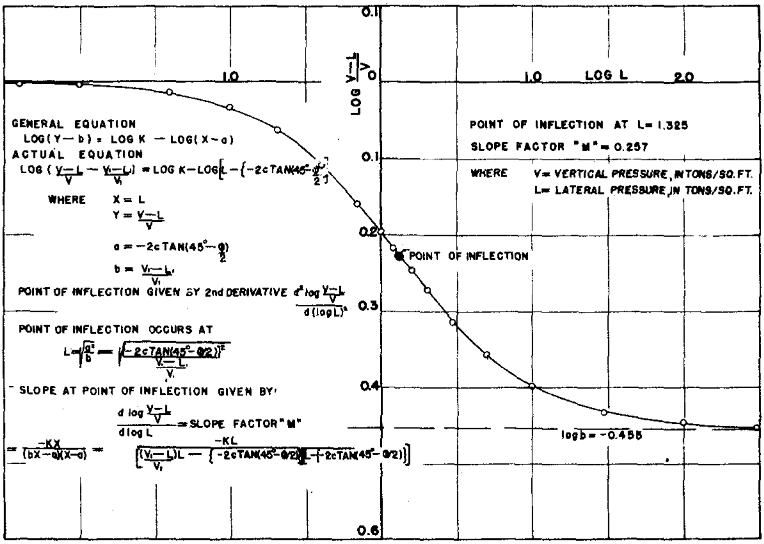


FIG. 33 LOG OF RATIO $\frac{V-L}{V}$ VERSUS LOG L FOR TRIAXIAL COMPRESSION TEST

It was also found that a rectangular hyperbola resulted when (V-L)/V was plotted versus V, Fig. 32, and this curve was represented by the general equation,

$$(x)\ (y - b) = K$$

where each symbol has the significance indicated in Fig. 32, and K is a constant.

It is obvious that there is nothing significant about the curves of Figs. 31 and 32. However, when $\log \left(\frac{V-L}{V}\right)$ is plotted versus log L, the reverse curve graph of Fig. 33 is obtained.

The graph of $\log \left(\frac{V-L}{V}\right)$ versus log V, on the other hand, is without special significance. It might also be added that graphs of (V-L)/L versus L, of (V-L)/L versus V, of $\log \frac{(V-L)}{L}$ versus log L and of $\log \left(\frac{V-L}{L}\right)$ versus log V, are likewise devoid of any significant feature.

It was thought that the slope of the reverse curve of Fig. 33 at the point of inflection might be the sought after definite quantitative value provided by the triaxial compression test, which could be correlated with the corresponding plate bearing test. The slope of the curve at the point of inflection in Fig. 33 is referred to in the next few paragraphs as "slope factor 'm'."

The slope of the tangent at any point on a curve is given by the first derivative of the equation for the curve. The lateral pressure "L" at which the point of inflection occurs in Fig. 33, is found by equating the second derivative of the equation for the curve to zero.

An outline of the equations and mathematical derivations involved in obtaining expressions for the values of the slope of the curve (slope factor "m") and of lateral pressure "L," at the point of inflection, is given in Fig. 33, and need not be repeated here.

From the mathematical equations derived in this manner, the value of slope factor "m" can be calculated for each Mohr diagram representative of the undisturbed samples from each test location. Fig. 34 is a graph of the values of slope factor "m" versus load test results on a 30-inch diameter plate at 0.2 inch deflection for the six airports with cohesive subgrade soils for which triaxial compression tests data were obtained. It is apparent that a relationship

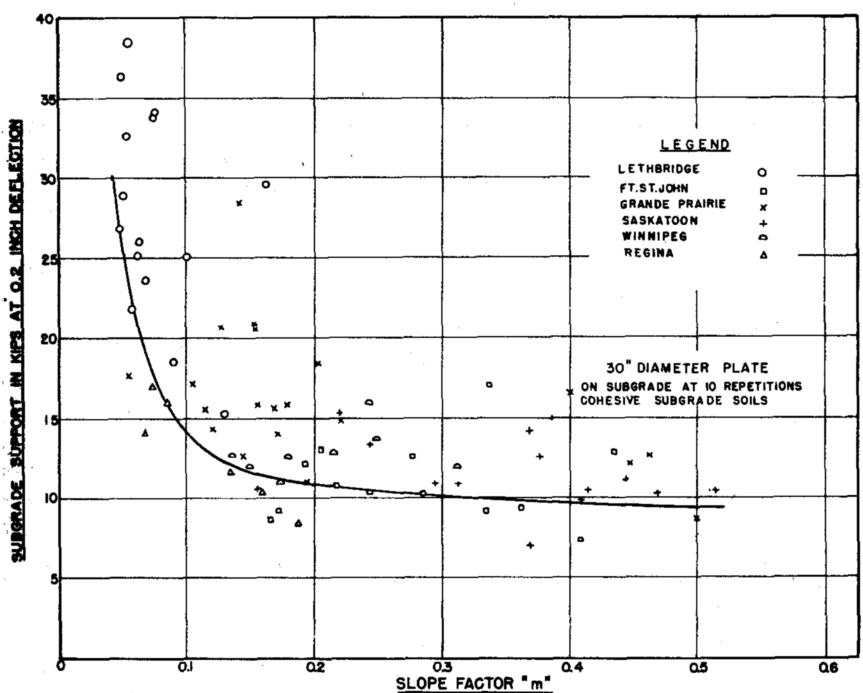


FIG.34 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS SLOPE FACTOR "m"

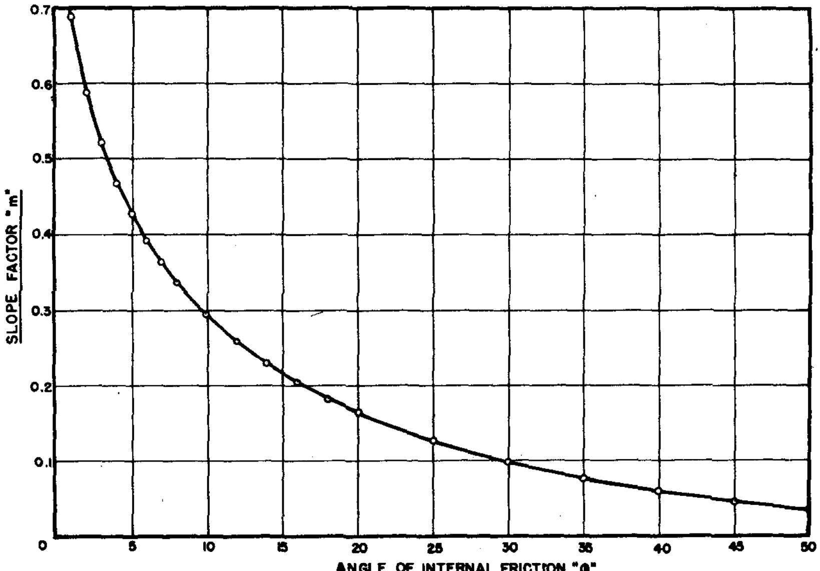


FIG. 35 SLOPE FACTOR "m" VERSUS ANGLE OF INTERNAL FRICTION "φ"

appears to exist, and a best average line can be drawn through the points as shown.

Further study of the mathematical equations indicated that slope factor "m" is independent of cohesion "c," and is entirely a function of the angle of internal friction "φ." The relationship between slope factor "m" versus angle of internal friction "φ" is shown in Fig. 35.

Consideration of the information in Figs. 34 and 35 implies that a relationship should exist between angle of internal friction "φ" and subgrade support on a 30-inch diameter plate at 0.2 inch deflection, Fig. 36.

An attempt was made to establish the best average line through the data of Fig. 36 on the usual basis that the subgrade bearing capacity indicated by the "φ" values curve would not exceed the true subgrade support on a 30-inch diameter plate at 0.2 inch deflection by more than about 10 per cent for any of the six airports. However, because of the distortion of the curve which this would have required, the deviation for Regina is about 15 per cent.

There is obviously no advantage in going through the mathematical calculations required to determine slope factor "m," in order to obtain the corresponding value of the subgrade support on a 30-inch diameter plate at 0.2 inch deflection, from Fig. 34, when the latter information can be

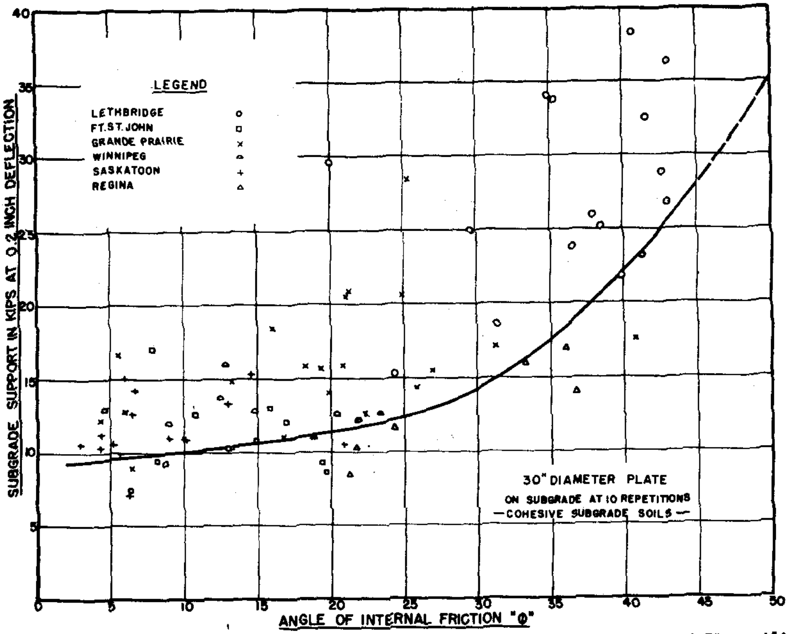


FIG. 36 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS ANGLE OF INTERNAL FRICTION "Ø"

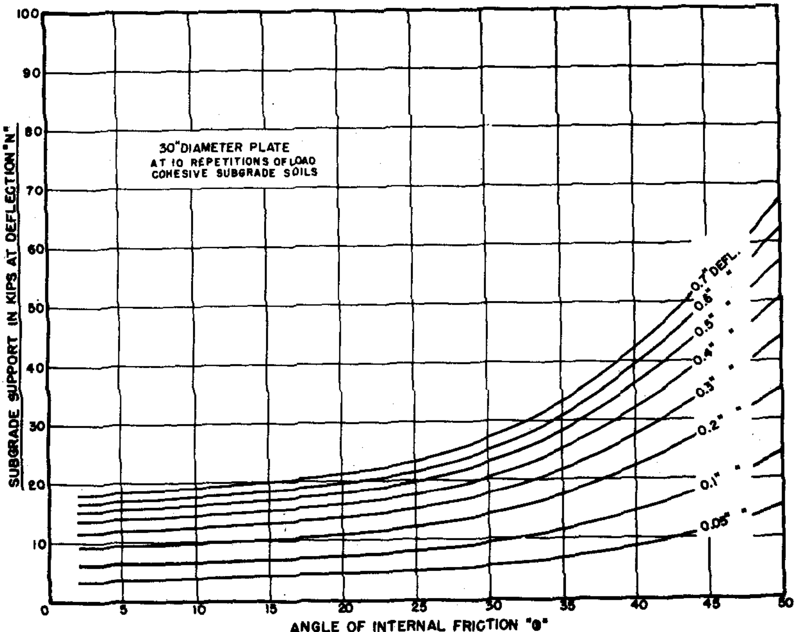


FIG. 37 SUBGRADE SUPPORT IN KIPS AT DEFLECTION "N" VERSUS ANGLE OF INTERNAL FRICTION "Ø"

determined with identical accuracy from Fig. 36 based upon the angle of internal friction "φ," which can be read off directly from the Mohr diagram.

Fig. 37 indicates the relationships for angle of internal friction "φ" versus subgrade support on a 30-inch diameter plate for deflections from 0.0 to 0.7 inch. Fig. 37 combines the information of Figs. 15 and 36. Information similar to that of Fig. 37 can also be prepared for Field C.B.R., cone bearing, and Housel penetrometer tests.

If "L_i" represents the lateral pressure at the point of inflection, Fig. 33, and "V_i" is the corresponding vertical pressure, it might be expected that relationships should exist between subgrade support on a 30-inch diameter plate at 0.2 inch deflection versus V_i, or versus $(V_i - L_i)$, or versus L_i, or versus $(V_i - L_i)/V_i$. Graphs of the data for these relationships are given in Figs. 38, 39, 40 and 41, respectively.

While a relationship might be particularly anticipated between subgrade support on a 30-inch plate versus V_i, the scattering of data in Fig. 38 indicates that none appears to exist. This is equally true when load test data are plotted versus $(V_i - L_i)$, Fig. 39.

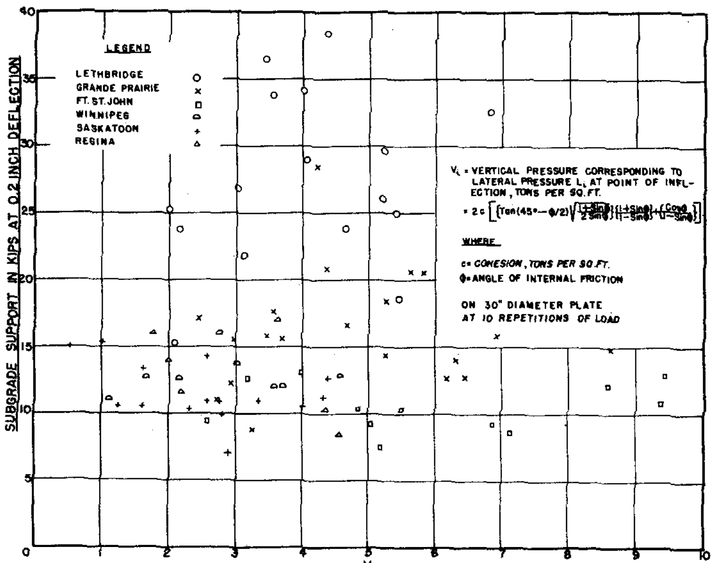


FIG. 38 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS V_i FOR TRIAXIAL COMPRESSION TEST.

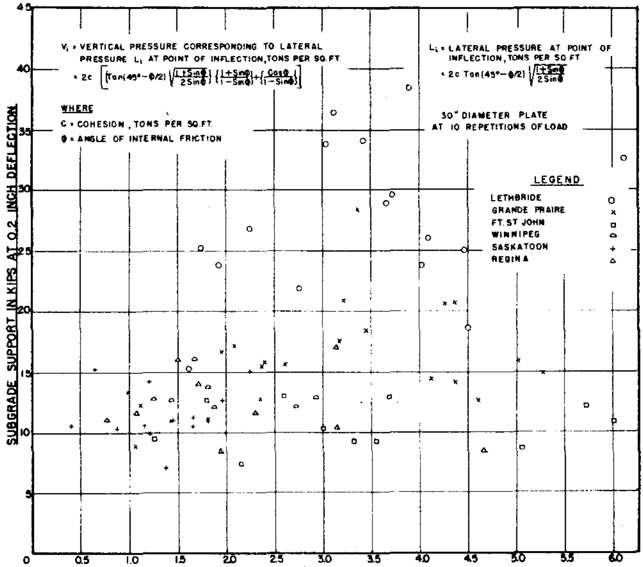


FIG.39 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS $V_i - L_i$ FOR TRIAXIAL COMPRESSION TEST.

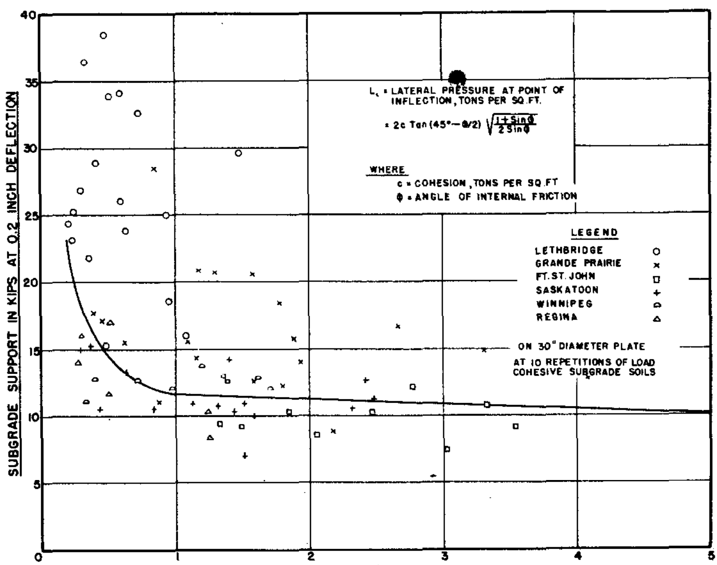


FIG.40 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS L_i FOR TRIAXIAL COMPRESSION TEST.

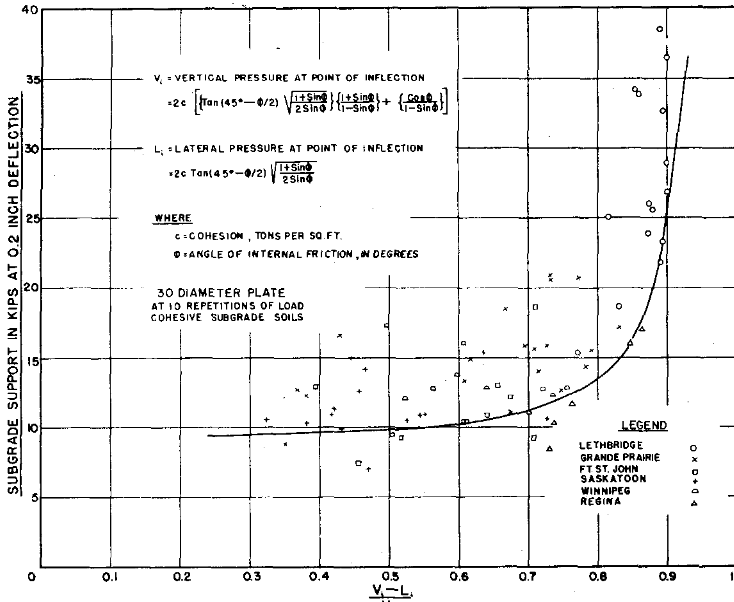


FIG. 41 SUBGRADE SUPPORT IN KIPS AT 0.2 INCH DEFLECTION VERSUS $\frac{V_i - L_i}{V_i}$ FOR TRIAXIAL COMPRESSION TEST

A reasonable graph is obtained when L_i is plotted versus subgrade support on a 30-inch diameter plate, Fig. 40, but because of the sharpness of the curve, and the flatness of the lower portion, it seems less satisfactory as a basis for design than the relationship between "φ" and load test data of Fig. 36. On the other hand, the graph of Fig. 40 has the advantage that L_i includes values of both "c" and and "φ" from any Mohr diagram, and it may be of particular advantage for soft cohesive soils with values of "φ" approaching zero, but having measurable values of "c."

A very good relationship exists between subgrade support on a 30-inch diameter plate versus $(V_i - L_i)/V_i$, Fig. 41. However, in any ratio such as V_i/L_i, or $(V_i - L_i)/V_i$, etc., the cohesion "c" term cancels from both numerator and denominator, and the expression is seen to be dependent on "φ" only. Therefore, $(V_i - L_i)/V_i$ is independent of cohesion "c" and is a function of "φ" only. Consequently, there is no advantage in determining the value of $(V_i - L_i)/V_i$ since a similar relationship with respect to load test values can be determined from φ, Fig. 36.

15. Cone Bearing, Housel Penetrometer, Field C.B.R. and Triaxial Compression Tests Compared.

In Table 6, a comparison is made between the load test data obtained indirectly from the best average line through the field C.B.R., cone bearing, Housel penetrometer, and triaxial compression tests data of Figs. 25, 27, 29, and 36, respectively, and the actual load test data provided by plate bearing tests for each of the eight airports with cohesive soils. Because of the basis on which their position was established (overestimate of subgrade support must not exceed about 10 per cent for any airport), it is obvious that the locations of the best average lines in Figs. 25, 27, 29 and 36, would fit the data for some airports better than for others. Table 6 indicates for each airport, the deviation between the load test information obtained indirectly from the correlation curves for these four tests, and the actual load test results determined by plate bearing tests. Percentages greater than 100 show that the best average curve for that test has overestimated the subgrade support for that airport by the amount of the difference between the percentage given and 100 per cent. Similarly, the subgrade support has been underestimated according to the best average line, wherever the percentage shown in Table 6 is less than 100 per cent.

In general, reasonably good agreement is indicated by Table 6, between the actual load test information and the subgrade support determined indirectly by means of the four tests. There are two or three airports in the case of each of these four tests, for which this agreement is poorer than for others, but in only one case is the deviation greater than thirty per cent, and in only five cases is it greater than twenty per cent.

From the right hand column of Table 6, it will be seen that when the results of the four tests are averaged, the actual load tests data are approximated within 10 per cent except for Lethbridge and Grande Prairie. Of even greater interest is the fact that if the cone bearing and Housel penetrometer test data in Table 6 are averaged, the results are within 10 per cent of the actual load test data for all airports except Lethbridge.

In Fig. 42, the cone bearing, Housel penetrometer, field C.B.R., and φ (from the triaxial compression test) values are plotted versus subgrade support on a 30-inch diameter plate at 0.2 inch deflection for cohesive subgrade soils. Consequently, Fig. 42 combines into one graph, the information provided by the best average lines through the data of Figs. 25, 27, 29 and 36.

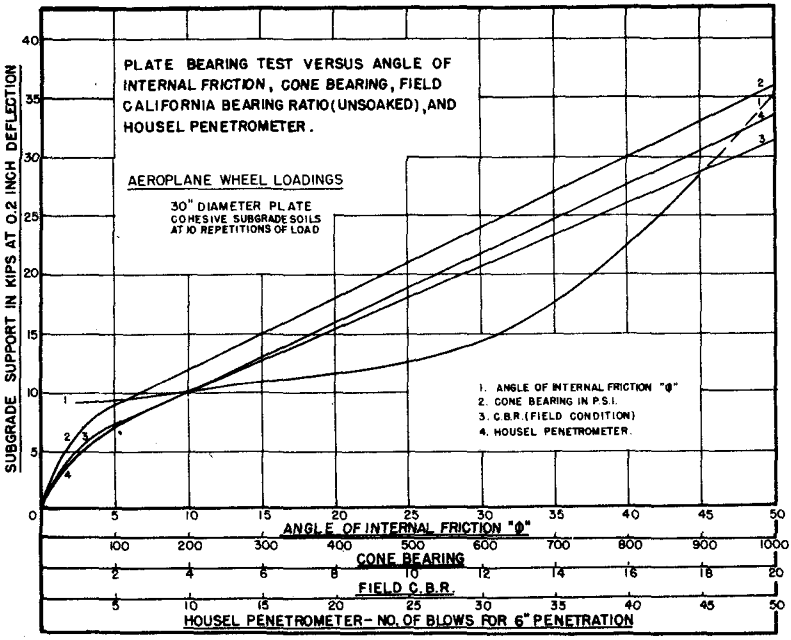


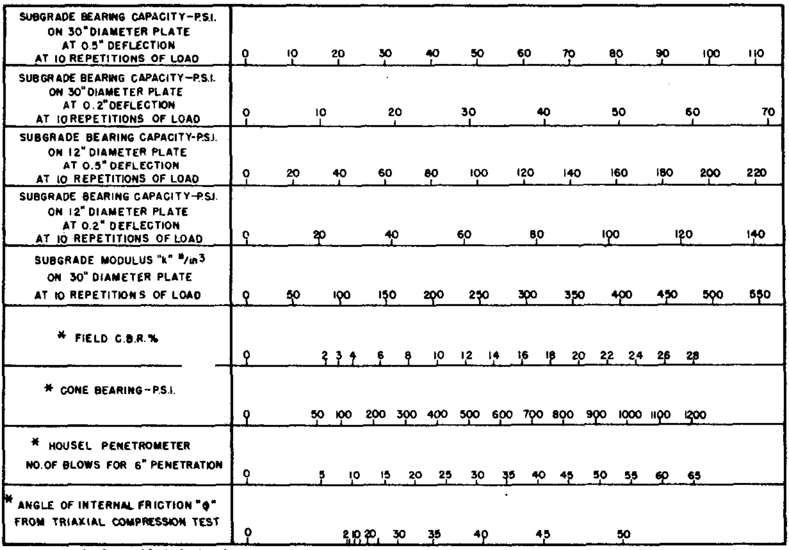
FIG. 42

Table 6

Ratio of load test values given by best average line through data for cone bearing, Housel penetrometer, field C.B.R., and triaxial compression tests, versus actual load test values given by plate bearing tests. Ratios expressed as percentages. Data for eight airports with cohesive subgrade soils.

Ratio of load test values read from best average line through data for the following tests versus actual load test values. Ratios expressed as percentages.

Airport	Cone Bearing	Housel Penetrometer	Field C.B.R.	Triaxial Compression	Overall Average
Fort St. John	108.1	98.7	109.5	93.3	102.4
Grande Prairie	90.0	96.6	82.2	76.2	86.3
Lethbridge	85.6	64.8	74.8	79.9	75.5
Saskatoon	109.7	109.8	104.6	89.5	103.4
Regina	85.7	101.9	115.9	113.7	109.3
Winnipeg	78.7	108.2	85.2	86.7	89.7
Toronto	103.8	90.9	85.1	-	93.3
Montreal	101.0	83.9	87.7	-	90.9



* Average value for top 2 feet of subgrade

FIG. 43 RELATIONSHIPS BETWEEN THE MEASUREMENTS OF SUBGRADE BEARING CAPACITY INDICATED BY VARIOUS TEST METHODS (FOR COHESIVE SUBGRADE SOILS)

Fig. 43 consists of a chart which provides relationships between plate bearing tests on a 30-inch plate at deflections of 0.2 and 0.5 inch, plate bearing tests on a 12-inch plate at 0.2 and 0.5 inch, subgrade modulus "k" determined with a 30-inch plate, and values of cone bearing, field C.B.R., Housel penetrometer, and triaxial compression tests, all with reference to 10 repetitions of load on cohesive subgrade soils.

16. Bituminous Mixture Design by the Triaxial Compression Test.

A further development of the information for the triaxial compression test presented in Section 14 above, appears to be applicable to the design of bituminous paving mixtures. Figs. 44 to 51 contain data leading up to this development.

Figs. 44, 45, 46 and 47 illustrate relationships involving L_1 and V_1, the lateral and vertical pressures respectively, corresponding to the point of inflection in Fig. 33.

Fig. 44 shows the relationship between L_1 versus φ for several values of cohesion "c."

In Fig. 45 the graph of V_1 versus φ is given for three values of cohesion "c." Like L_1, the value of V_1 depends upon both φ and "c."

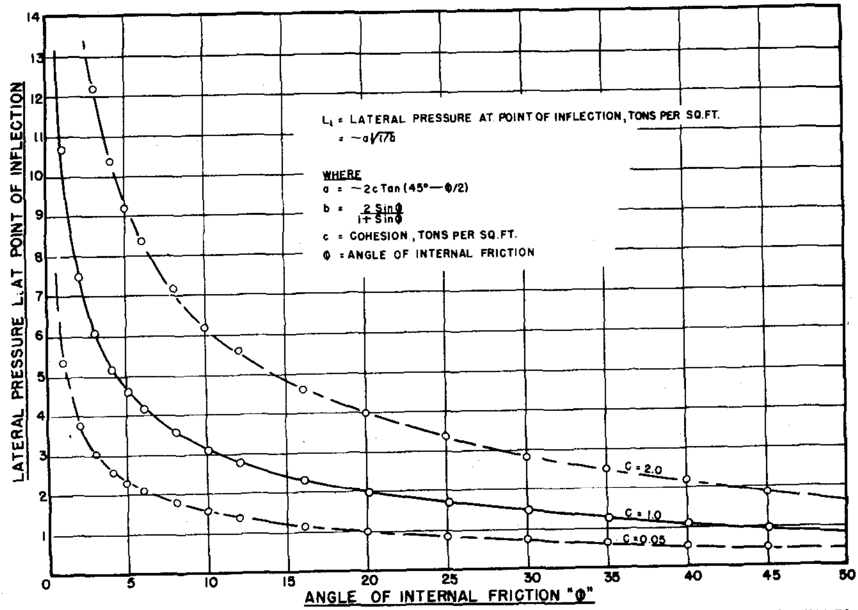


FIG. 44 LATERAL PRESSURE L_i VERSUS ANGLE OF INTERNAL FRICTION Φ FOR VARIOUS VALUES OF COHESION "c" FOR TRIAXIAL COMPRESSION TEST

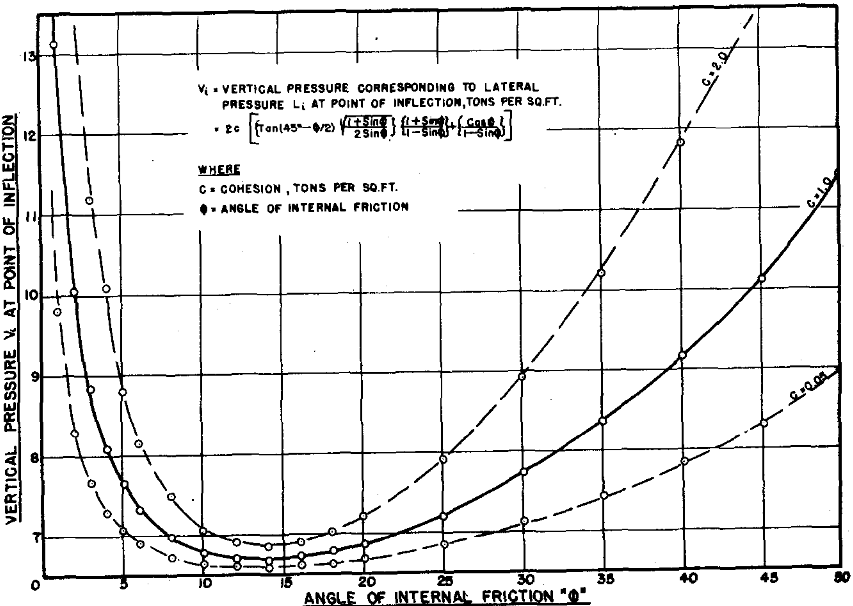


FIG. 45 VERTICAL PRESSURE V_i VERSUS ANGLE OF INTERNAL FRICTION Φ FOR VARIOUS VALUES OF COHESION "c" FOR TRIAXIAL COMPRESSION TEST.

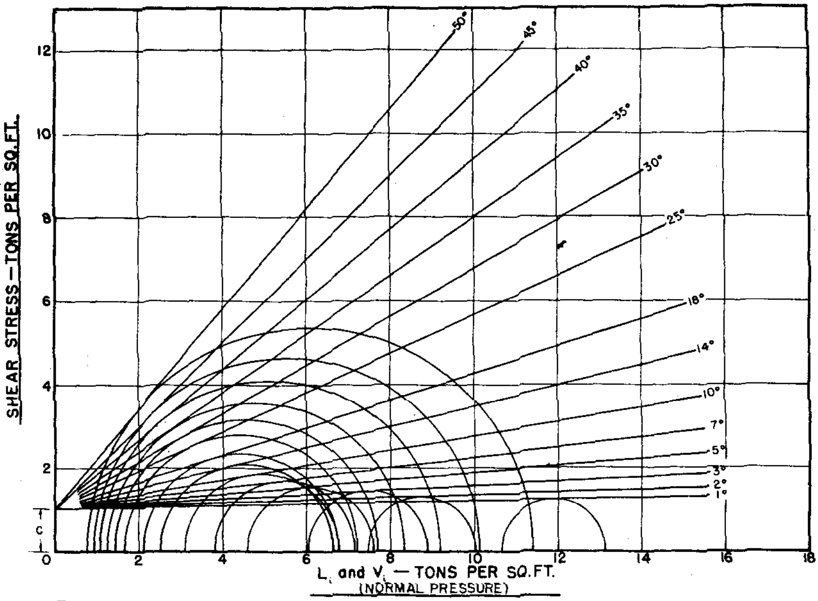


FIG. 47 MOHR DIAGRAM ILLUSTRATING INFLUENCE OF VARIATION IN ANGLES OF INTERNAL FRICTION "ϕ" ON VALUES OF L_i AND V_i WHEN COHESION "c" IS CONSTANT.

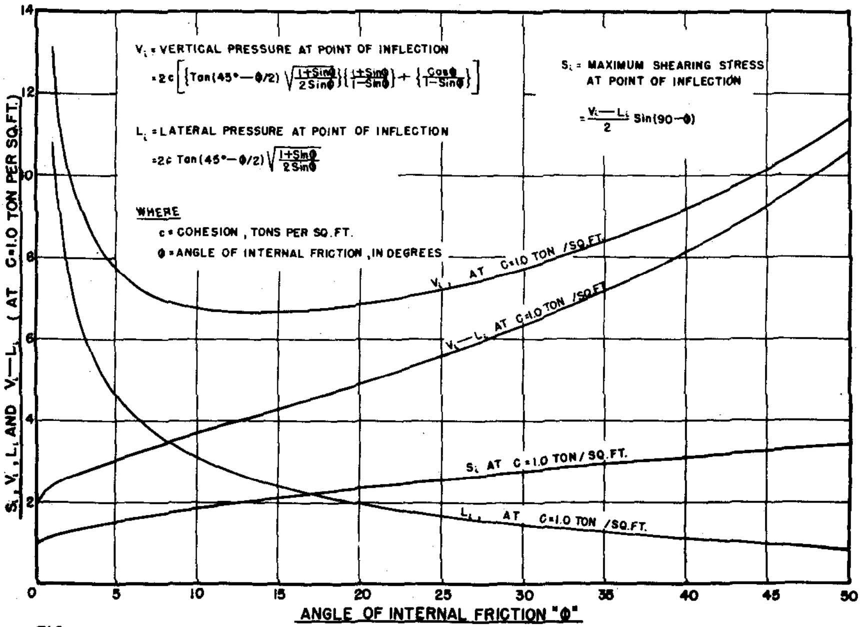


FIG. 46 S_i, V_i, L_i, AND $V_i - L_i$ VERSUS ANGLE OF INTERNAL FRICTION "ϕ" WHEN COHESION c=1.0 TON/SQ.FT

The equation for V_1 can be expressed somewhat more simply than that given on the various figures, as shown below,

$$V_1 = 2c \frac{\cos\varphi}{1 - \sin\varphi} \left(\frac{1 + \sin\varphi + 1}{2\sin\varphi}\right)$$

In Fig. 46, S_1, L_1, V_1, and $(V_1 - L_1)$ are plotted versus φ for a value of cohesion "c" equal to unity in each case. S_1 is the maximum shear stress at the point of inflection.

Figs. 44 and 46 indicate that when "c" is constant, for each value of L_1 there is only one corresponding value of φ. Figs. 45 and 46, on the other hand, show that when "c" is constant, for each value of V_1 there are two corresponding values of φ, provided φ is greater or less than 13° 39' 16.7", the value of φ at which the minimum value of V_1 occurs.

When "c" is constant, it is of interest that while L_1 decreases continually as φ increases, V_1 decreases as φ increases until it reaches the value of 13.7° after which V_1 increases as φ increases. The reason for this can be readily seen by reference to Fig. 47, which is a Mohr diagram in terms of L_1 and V_1, with c constant but φ variable.

Cohesion "c" in Fig. 47 has the value of unity and Mohr rupture lines have been drawn for values of the angle of internal friction φ varying from zero to greater than 45° When φ is zero, both V_1 and L_1 are infinitely great and are therefore off the diagram to the right. For values of φ from 0° to 13.7°, both V_1 and L_1 steadily decrease as φ increases. For all values of φ greater than 13.7°, L_1 continues to decrease, while V_1, on the other hand, begins and continues to increase as φ increases from 13.7° to 90°. Consequently, for any given value of cohesion "c," the lowest value of V_1 occurs at an angle of 13.7°, and V_1 increases as φ decreases toward 0 or increases toward 90° from this critical value of 13.7°.

Fig. 47 demonstrates that for a constant value of c, the value of $(V_1 - L_1)$ decreases as φ decreases, and vice versa. It can be similarly shown that for a constant value of φ, $(V_1 - L_1)$ decreases as c decreases, and vice versa. Consequently, $(V_1 - L_1)$ values represent a measure of the stability of a cohesive material being tested by triaxial compression.

Fig. 46 indicates that $(V_1 - L_1)$ increases as φ increases, when "c" is constant, but a point of inflection

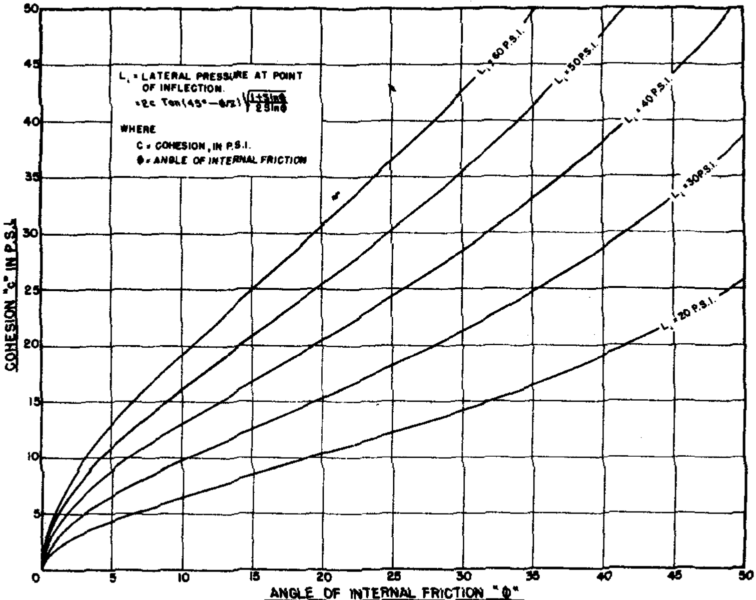


FIG. 48 RELATIONSHIPS BETWEEN COHESION "c", ANGLE OF INTERNAL FRICTION "Φ", AND LATERAL PRESSURE AT THE POINT OF INFLECTION "L_i" FOR TRIAXIAL COMPRESSION TEST

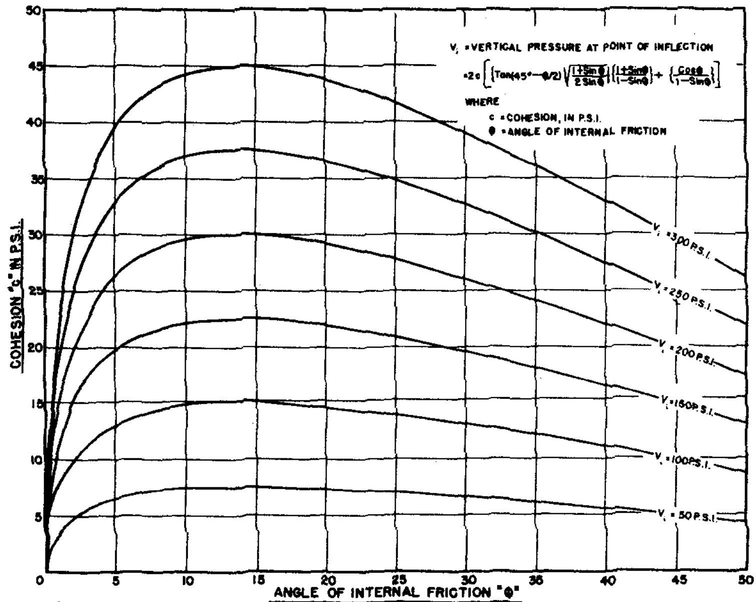


FIG. 49 RELATIONSHIPS BETWEEN COHESION "c", ANGLE OF INTERNAL FRICTION "Φ", AND VERTICAL PRESSURE AT THE POINT OF INFLECTION "V_i" FOR TRIAXIAL COMPRESSION TEST.

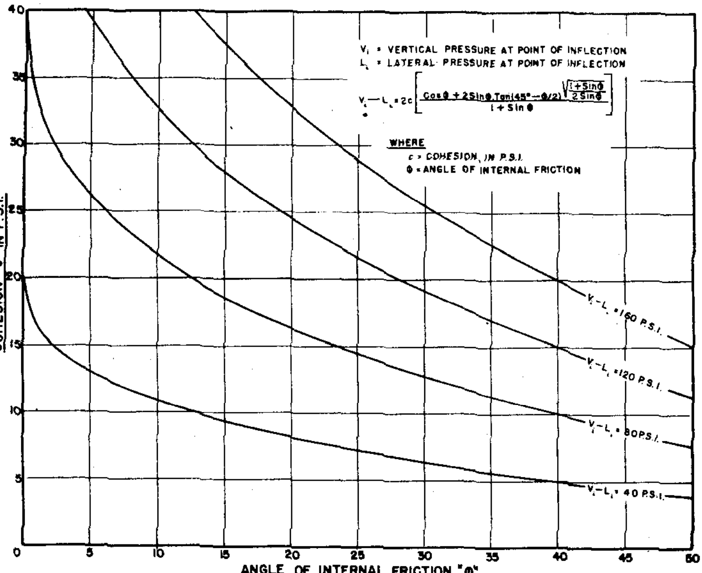


FIG. 50 RELATIONSHIPS BETWEEN COHESION 'c', ANGLE OF INTERNAL FRICTION "ϕ", AND $V_i - L_i$ FOR TRIAXIAL COMPRESSION TEST

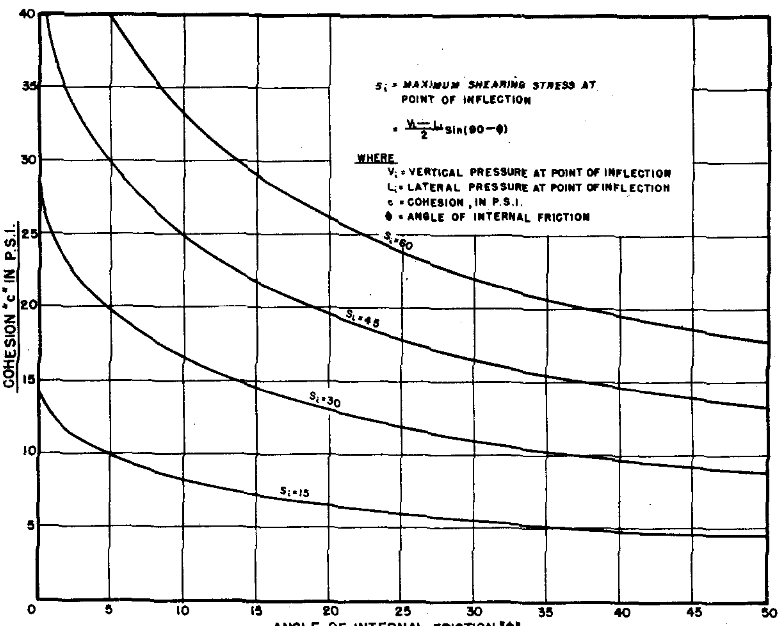


FIG. 51 RELATIONSHIPS BETWEEN COHESION "c", ANGLE OF INTERNAL FRICTION "ϕ", AND S_i FOR TRIAXIAL COMPRESSION TEST.

occurs when φ is equal to 13.7°. S_1 increases slowly as φ increases over the range of 0 to 50°.

Figs. 48, 49, 50 and 51, are graphs of cohesion "c" versus angle of internal friction "φ," for values of L_1, V_1, $(V_1 - L_1)$, and S_1, respectively. Fig. 48 indicates that the curve for each value of L_1 rises as "c" and φ are both increased, and a point of inflection occurs when $\varphi = 13.7^\circ$.

Fig. 49 shows that the curve for each value of V_1 rises as "c" and φ increase, over the range of φ between 0° and 13.7°. Thereafter the curve falls as φ increases beyond 13.7°.

Fig. 50 indicates that the curve for any given value of $(V_1 - L_1)$ falls steadily as φ increases from 0° to 50° and beyond.

From Fig. 51, it is seen that the curve for any value of S_1, the maximum shear stress at the point of inflection, falls steadily as φ increases from 0° to 50°.

Fig. 52 is a chart taken from a manual on the design of asphaltic concrete recently published by the Asphalt

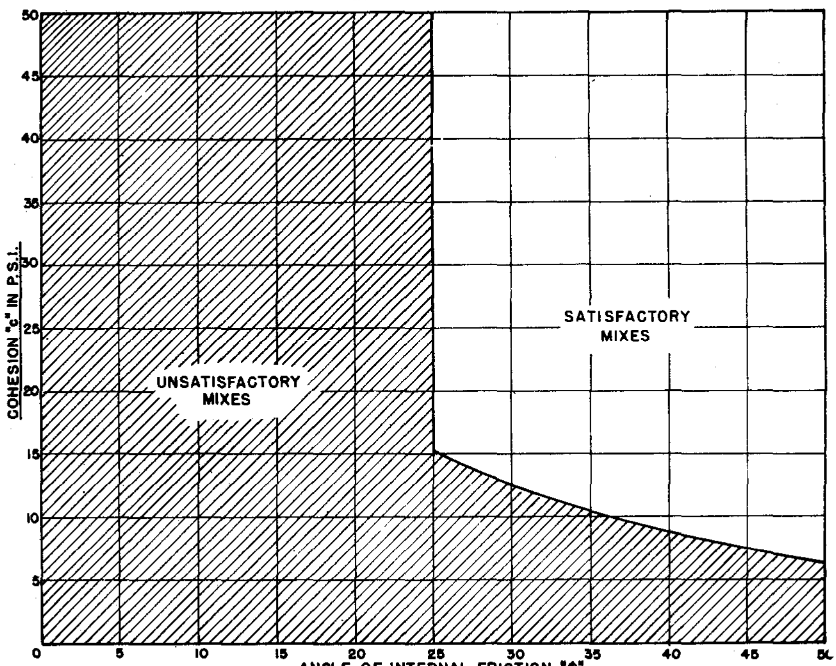


FIG.52 DESIGN CHART FOR ASPHALTIC CONCRETE BASED UPON THE TRIAXIAL COMPRESSION TEST (THE ASPHALT INSTITUTE MANUAL ON HOT-MIX ASPHALTIC CONCRETE PAVING)

Institute[22]. The unshaded portion of this chart indicates the corresponding ranges of values for "c" and φ which asphaltic concrete mixtures must possess according to the manual, for satisfactory stability and performance when they are designed by the triaxial compression test. The cross-*hatched area of the chart represents those combinations of* "c" and φ which are reported to result in the poor behaviour of asphaltic concrete pavements.

The single boundary of the diagram of Fig. 52 between satisfactory and unsatisfactory combinations of c and φ for bituminous mixture design is illogical, since for certain projects the stability requirements given by this boundary will be too high, and for other projects will be too low. The diagram would have greater value if it were zoned into low, moderate, and high stability requirements.

Fig. 53 represents a combination of the information of Figs. 50 and 52. The $(V_i - L_i)$ curves of Fig. 50 have been superimposed upon the asphaltic concrete design chart of Fig. 52. *It will be observed that a* $(V_i - L_i)$ *value of*

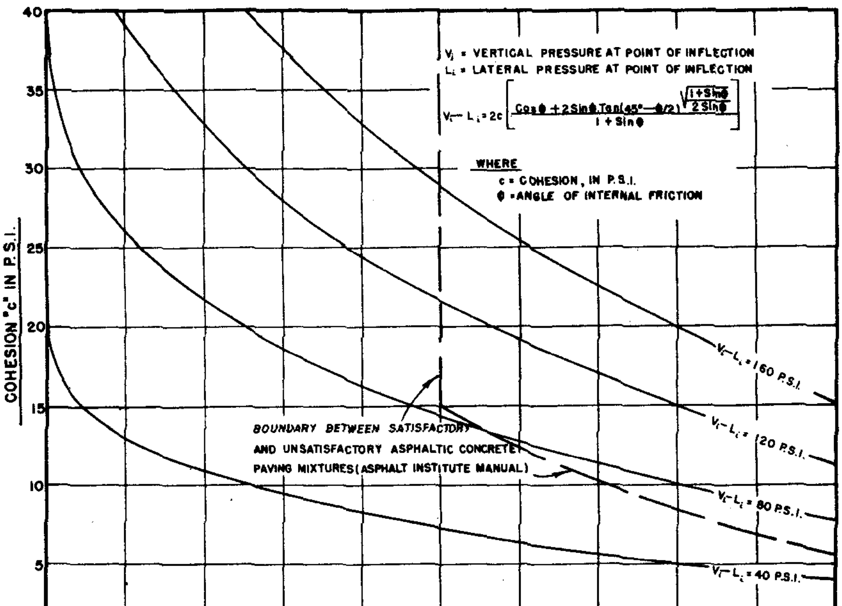


FIG.53 GRAPH SHOWING THE BOUNDARIES BETWEEN SATISFACTORY AND UNSATISFACTORY ASPHALTIC CONCRETE MIXTURES PROPOSED BY THE ASPHALT INSTITUTE, AND $V_i - L_i$ VALUES.

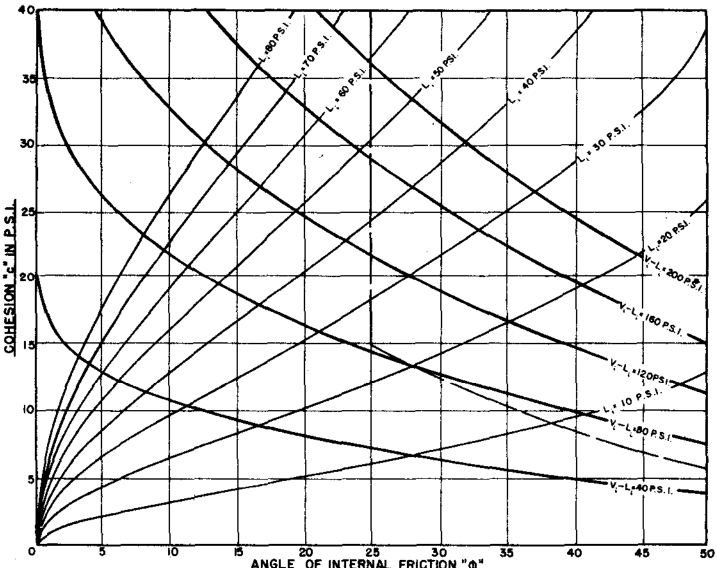


FIG.54 CHART FOR ASPHALTIC CONCRETE DESIGN BASED UPON VALUES OF c, Φ, L_i, AND $V_i - L_i$ DERIVED FROM THE TRIAXIAL COMPRESSION TEST.

80 p.s.i. coincides very well with the lower boundary of Fig. 52. Curves representing lower values of (V_i - L_i) than this lie within the portion of the chart labelled unsatisfactory. (V_i - L_i) curves for higher values than 80 p.s.i., and to the right of the $\varphi = 25^\circ$ ordinate, lie within the area of the chart considered to represent satisfactory design.

Fig. 54 combines the information of Figs. 48 and 53, and superimposes graphs for several (V_i - L_i) and L_i values respectively, on the Asphalt Institute diagram of Fig. 52.

Every bituminous paving mixture can mobilize only so much lateral support against displacement by applied vertical loads. For weak mixtures, the inherent lateral support that can be mobilized is probably moderate and for strong bituminous mixtures it may be considerably greater. The exact amount of lateral support inherently available within each bituminous pavement in place is unknown at present in quantitative terms, but might be determined from its performance under traffic, coupled with triaxial compression test studies of samples from the pavement.

Figs. 53 and 54 indicate that stability requirements for bituminous paving mixtures could be very satisfactorily zoned in terms of (V_i - L_i) values. For example, a (V_i - L_i) value of 80 p.s.i. might be adequate where average stability was required. A (V_i - L_i) value of 120 p.s.i. might however be required wherever there was much starting and stopping of motor vehicles, as at stop lights, bus stops, etc., while for pavements for secondary roads, a $(V_1 - L_1)$ value of 40 p.s.i. or less might provide adequate stability under traffic. The $(V_1 - L_1)$ value representing the minimum stability that could be tolerated for the pavement on a given project could be specified, and the curve for this value would establish the boundary between satisfactory and unsatisfactory stabilities for that project. The only limitation on this boundary, ($(V_1 - L_1)$ curve), would exist on the left hand side, and would be marked by the intersection of the $(V_1 - L_1)$ curve, with the curve for the L_1 value corresponding to the maximum lateral support that could be mobilized within the pavement in place under expected traffic loads when in its most critical condition, (probably its highest summer temperature). This L_1 value might frequently be to the left of the vertical boundary shown in the Asphalt Institute diagram, Fig. 52. The critical L_1 value could be expected to vary from project to project depending upon the maximum lateral support that could be mobilized within the pavement in place, and it would probably also vary with the $(V_1 - L_1)$ value specified or adopted for the paving mixture.

Bituminous mixtures having combinations of c and φ (giving higher L_1 values) to the left of this critical L_1 value for each bituminous pavement in place, would be satisfactory in themselves, but would tend to be unsatisfactory in service for the project in question in each case, because they could not mobilize sufficient lateral support to develop the minimum $(V_1 - L_1)$ value specified for stability. The justification or otherwise for sharply defining the critical value of L_1 to be adopted for each paving project can only be established as the result of considerable investigation, since calculations indicate that a considerable decrease in lateral support does not markedly or rapidly lower the $(V - L)$ stability value of the paving mixture. That is, the stability of a bituminous mixture in place does not appear to be critical with regard to appreciable changes in the amount of lateral support which it can mobilize under traffic loads. The stability appears to depend much more critically upon changes in the $(V_1 - L_1)$ values specified for design, than upon modifications in the degree of inherent lateral support that can be developed by the pavement.

It is believed that L_1 and $(V_1 - L_1)$ curves similar to those of Fig. 54, obtained from the triaxial compression test, represent a logical and useful method for the design of the stability requirements for bituminous paving mixtures. However, considerable investigation is needed to measure the maximum values of lateral support L_1 that can be developed, and the corresponding minimum $(V_1 - L_1)$ values required for stability, under different magnitudes of wheel load and various intensities of traffic. This involves observation of the performance under traffic of various bituminous surfaces having a wide range of c and φ values, and the study of samples of these pavements by means of the triaxial compression test.

It should be noted that a diagram somewhat similar to Fig. 54 for designing the stability for bituminous mixtures could also be prepared on the basis of S_1 values, Fig. 55.

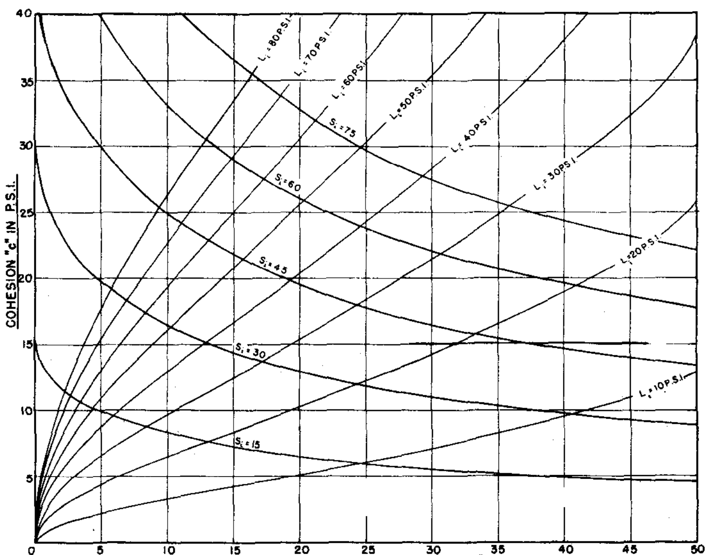


FIG. 55 CHART OF RELATIONSHIPS BETWEEN S_1, L_1, c, AND φ FROM THE TRIAXIAL COMPRESSION TEST

17. Selection of Base Course Materials by the Triaxial Compression Test.

It was pointed out in Section 9, that for similar relative density and moisture conditions, different types of granular bases may have the same supporting capacity per unit of thickness. If this is substantiated by further investigation, it means that there is little or no difference in the ability of a given thickness of different types of granular bases to distribute an applied load over the subgrade. A much wider range of granular materials, including sands, might therefore function satisfactorily as base courses than it has been considered advisable to use in the past.

In addition to providing the necessary thickness however, a base course material must be able to develop adequate shear resistance against the shearing stresses imposed by the applied load. The highest shear stresses occur nearest the loaded area, and it is for this reason that the best granular materials are usually specified for the top layer of the base course. If the base course has sufficient thickness, the shearing resistance of the subgrade will not be exceeded.

In Fig. 56, the shear stress trajectories under a loaded area are indicated. If the shearing resistance along the full line in Fig. 56 were exceeded, base course material under load would move laterally and upward along this trajectory, leading to rutting and probable failure.

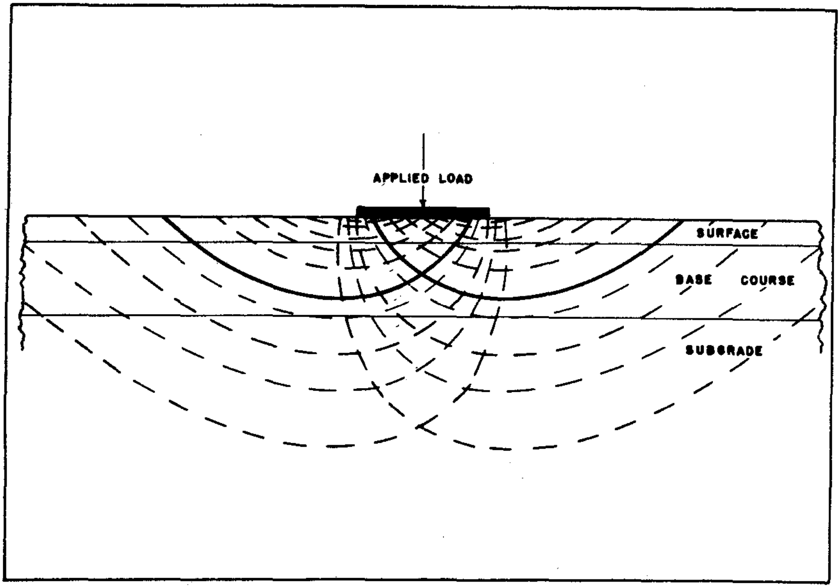


FIG.56 DIAGRAM OF SHEAR STRESS TRAJECTORIES UNDER A LOADED AREA

The shearing resistance of base course materials can be determined by the triaxial compression test. Those with measureable cohesion can be evaluated by means of Fig. 55, in which the different degrees of stability under load are zoned in terms of S_1 and L_1 values. For projects or portions of the base course subject to large shear stresses, a base course material with a high value of S_1 should be specified, and for locations where shear stresses are lower, a material with a lower S_1 value could be stipulated. Each S_1 curve in Fig. 55 would be bounded toward the left hand side by the L_1

curve corresponding to the maximum lateral support that could be mobilized within the base course in place under traffic loads, when in its most critical condition, (probably the highest moisture content anticipated).

Base course materials having combinations of c and φ giving the higher L_1 values to the left of this critical L_1 value for each base course in place, would tend to be unsatisfactory in service, because they could not mobilize sufficient lateral support to develop the minimum S_1 value specified for stability. However, calculations indicate that a considerable decrease in lateral support does not markedly or rapidly lower the S_1 stability value of a base course material possessing measureable cohesion. That is, the stability appears to be much more critical with regard to changes in the S_1 values specified for design, than upon modifications in the degree of inherent lateral support that can be mobilized within the base course (with cohesion) in place.

The stability requirements for base course materials with measureable cohesion could also be zoned in terms of $(V_1 - L_1)$ values, Fig. 54, after the manner described for bituminous mixtures in Section 16 above.

For base course materials having no measureable cohesion, Fig. 55 could not be employed. The shearing resistance of these as given by the Coulomb equation

$$s = n \tan \varphi$$

depends upon the normal pressure n and the angle of internal friction φ. For any given highway or airport project and wheel loading, the normal pressure (from load, confining influence, etc.) on the base course might be considered to be constant and independent of the nature of the granular base course material, as a first approximation. The shear strength or stability of granular base courses without cohesion would then vary directly with the magnitude of the angle of internal friction φ of the various materials. This angle can be determined by means of the triaxial compression test.

A study should therefore be made to determine the S_1 and L_1 values for base course materials with cohesion, and the values of φ for base course materials without cohesion, that are required for resisting the base course shearing stresses developed under different magnitudes of wheel load, and various intensities of traffic. This would involve observation of the performance under traffic of various base course materials having a wide range of c and φ values, and the investigation of samples of these materials by means of the triaxial compression test.

There was no evidence of base course shear failure at any of the airports tested so far, in spite of the different base course materials employed. Consequently, a much wider range of granular materials may function satisfactorily as base courses under the various range of loadings and traffic to which highways and airport runways are subjected, than is favoured at the present time. This applies particularly to sands and poorly graded gravels, which suitable tests might indicate are either satisfactory by themselves, or that they would be after admixture with a different granular ingredient, a filler, or other inexpensive material.

Evaluating in a quantitative manner, the requirements of granular materials for base courses, and the various inexpensive methods for improving the performance of otherwise unsatisfactory granular materials, has not received the research which the economic importance of base course materials to highway and airport engineers both justifies and demands.

18. Evaluation of Load Test Data for Flexible Pavement Design.

In Fig. 57 the surface load carried by a 30-inch diameter plate at 0.5 inch deflection has been plotted versus subgrade support on a 30-inch diameter plate at 0.5 inch at the same test location, for all load test sites on the runways at the eight airports with cohesive subgrade soils. Since the thickness of base course and wearing surface varied from about 6 inches to about 24 inches at the different test locations, the data of Fig. 57 have been corrected to an overall thickness of 12 inches on a simple proportionate basis.

Line "B" of Fig. 57 has been drawn at an angle of 45°, and represents on either axis, the load carried by the unconfined subgrade on a 30-inch diameter plate at 0.5 inch deflection.

Line "C" represents the best average line through the points of Fig. 57 and indicates, on the ordinate axis, the load carried by a 30-inch diameter plate on the surface of the runway (corrected thickness 12 inches) at 0.5 inch deflection versus the corresponding subgrade support at 0.5 inch deflection on the abscissa.

Line "Q" has been drawn parallel to line "B" and has the significance indicated below.

The following comments are made with regard to Fig. 57:

(1) In general, the points fall along a straight line, line C, passing through the origin.

(2) If the points had fallen along line B in Fig. 57, it would have meant that the base and wearing surface contributed

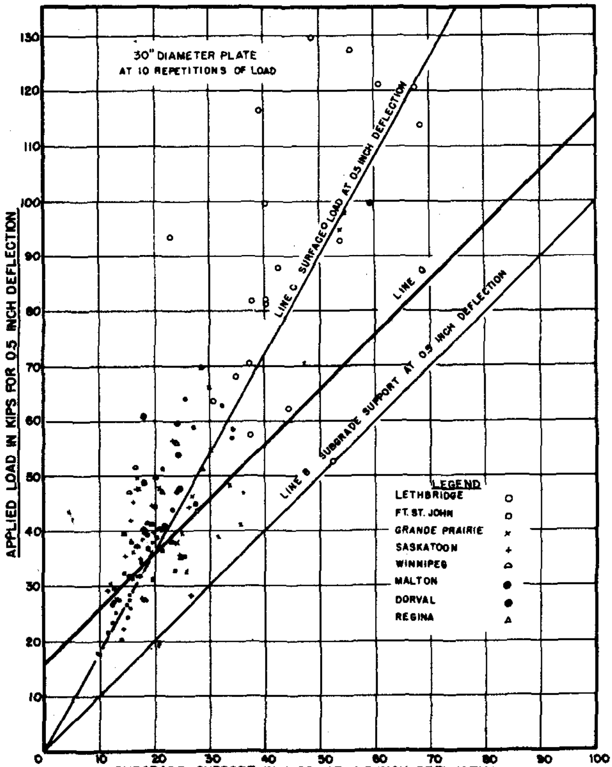


FIG.57 APPLIED LOAD VERSUS SUBGRADE SUPPORT FOR OVERALL THICKNESSES CORRECTED TO 12"

nothing to the load supporting capacity of the structure. That is, the load carrying capacity at 0.5 inch deflection, would have been no higher than that of the subgrade at this deflection. This, of course, would not be expected.

(3) Line Q indicates a location of the best average line through the points, that might have been expected on the assumption that 12 inches of a given base and surface would increase the load carrying capacity of a runway by exactly the same amount, regardless of the strength of the subgrade. That is, if 12 inches of a given base and surface increased the overall carrying capacity of a runway by 16,000 pounds when the subgrade support at 0.5 inch was 20,000 pounds, line Q indicates that this 12 inches of base and surface would likewise increase the overall carrying capacity by 16,000 pounds whether the

subgrade support were only 10,000 pounds, or 5,000 pounds, or 40,000 pounds, or any other value. Most of the theories and equations proposed in the past for the required thickness of flexible pavements are implicitly or explicitly based upon this assumption.

Fig. 57 demonstrates very definitely, however, that this is not the case, for there is no tendency for the points to fall along line Q, or along any other line parallel to line B.

On the other hand, Fig. 57 indicates very clearly that the points tend to fall along line C which passes through the origin.

(4) The most notable conclusion to be drawn from Fig. 57, is that <u>the increase in overall load carrying capacity provided by any given thickness of flexible base and surface varies directly with the load supporting value of the subgrade upon which it is placed</u>, when the bearing capacity of both subgrade and pavement are measured at the same deflection by bearing plates of the same diameter.

This conclusion appears to be reasonable after studying Fig. 57 since line B and line C both start from the origin and diverge instead of running parallel.

Consequently, if 12 inches of a given flexible base and wearing surface adds 16,000 pounds to the carrying capacity of a subgrade that supports 20,000 pounds at 0.5 inch deflection, the same thickness of base and surface will add 32,000 pounds to the carrying capacity of a subgrade supporting 40,000 pounds, but only 8,000 pounds to the carrying capacity of a subgrade supporting 10,000 pounds.

This means that the load carrying capacity of a given thickness of base and surface is doubled if the subgrade support is doubled, but is halved when the subgrade support becomes only one-half as great, etc.

(5) Fig. 57 emphasizes the value of increasing the strength of the subgrade under flexible pavements. Not only is the load supporting capacity of the subgrade itself increased, but the load carrying value of the base and surface per unit of thickness varies directly with the strength of the subgrade, and doubles when the subgrade support is doubled.

The work of other investigators has been studied for confirmation, or otherwise, of the principal conclusion to be drawn from Fig. 57, that the load carrying capacity of a given thickness of base and pavement varies directly with the strength of the subgrade upon which it is placed. It has been demonstrated elsewhere[20] that indications of agreement can be obtained from the investigations of others[8,10,21].

It would be interesting to know whether or not a somewhat similar conclusion could be developed from plate bearing tests on rigid pavements (particularly of the unreinforced type) and on the underlying subgrades, or if it is restricted to flexible pavements and bases. No data are available from this investigation to check the possible application of this conclusion to rigid pavement design.

19. Influence of Bearing Plate Size on Surface Load Versus Subgrade Support.

In Fig. 58, the influence of the size of bearing plates 30 inches and 12 inches in diameter on the ratio of

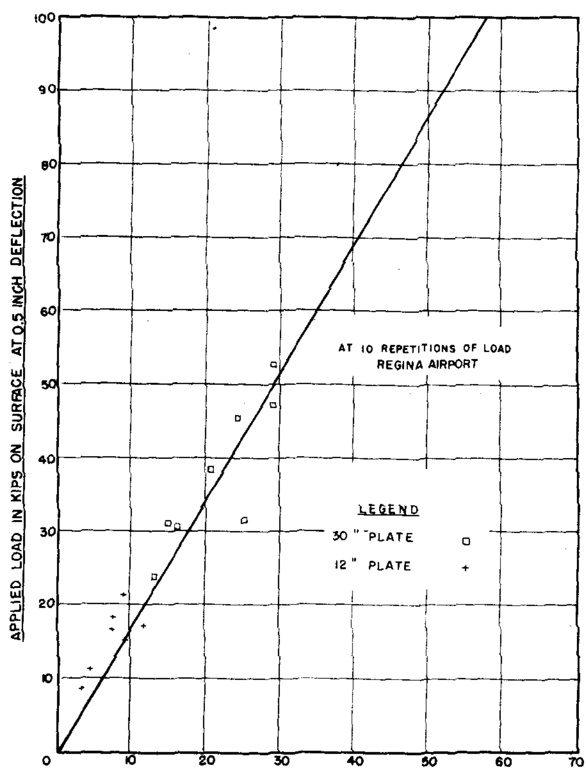


FIG.58 TOTAL APPLIED LOAD VERSUS SUBGRADE SUPPORT FOR BEARING PLATES OF DIFFERENT SIZES.

the load carried by the surface at 0.5 inch deflection versus that supported by the subgrade at 0.5 inch deflection is given for Regina Airport. Fig. 58 indicates that the best average line through the origin, and through the points for the 30-inch plate, is also quite representative for the location of the points for the 12-inch plate. This means that the relationship between surface load and subgrade support as determined by a 12-inch plate is the same as that which would have been indicated by the 30-inch plate for a weaker subgrade, other conditions remaining the same.

Data similar to those of Fig. 58 are provided by Fig. 59 for Lethbridge airport for bearing plates 12, 18, 24, 30, 36 and 42 inches in diameter. Again the best average line through the origin and through the points for the

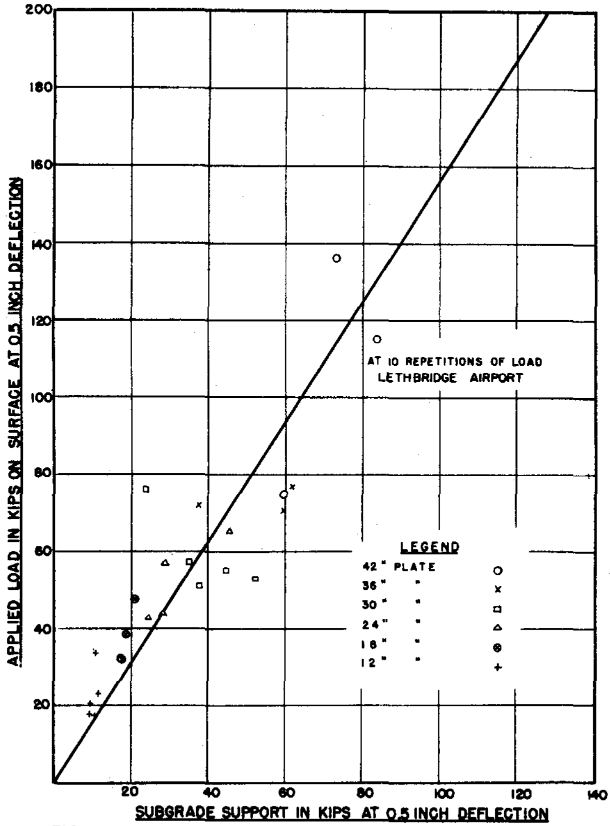


FIG.59 TOTAL APPLIED LOAD VERSUS SUBGRADE SUPPORT FOR BEARING PLATES OF DIFFERENT SIZES.

30-inch plate is, generally speaking, a reasonably representative line through the points for the other bearing plate sizes also. Consequently, the relationship between surface load and subgrade support determined by bearing plates of different sizes, corresponds to that for a 30-inch diameter plate for the same pavement and base, but on a stronger subgrade for larger plate sizes or on a weaker subgrade for smaller plate sizes. Or expressed somewhat differently, the relationship between surface load and subgrade support established by a 30-inch diameter plate holds for bearing plates of different diameters all other conditions being the same.

Confirmation of this trend has been developed[20] from some data of Campen and Smith[9].

20. A Method of Design for Thickness of Flexible Pavements.

One of the principal objectives of the Department of Transport's investigation was to develop a method of design based upon the load test data which had been obtained, that could be employed with reasonable confidence to establish the overall thickness of flexible base and surface required to carry aeroplane wheel loadings of any magnitude.

From the load test information obtained for each of the airports included in this investigation, an estimate of the load supporting capacity of the existing runways at each airport can be very easily made. For heavier wheel loads than these, greater thicknesses of pavement and base course are required. The problem of design therefore, consists of determining a method for extrapolating the test data obtained for the present runways, which will indicate the thicknesses required for aeroplane wheel loads of any magnitude.

It was demonstrated in Fig. 57, that the carrying capacity of a given thickness of base course material varies directly with the strength of the subgrade upon which it is placed. This observation can be developed into an equation for designing the thickness of flexible base and surface required to carry a wheel load of any magnitude.

Fig. 60 can be used to indicate the fundamental principle of this method of design. The load carrying capacity at 0.5 inch deflection of the first 6-inch layer of base course in Fig. 60, is normally greater than that measured on the surface of the subgrade at the same deflection. If the first 6-inch layer of base course is now considered to be the subgrade for the second 6-inch layer, all other conditions being the same, the load carrying capacity of the second layer will be greater than that of the first, since it rests on a stronger subgrade. Similarly, the load supporting capacity of a third 6-inch layer of base would be greater than that of the second, etc.

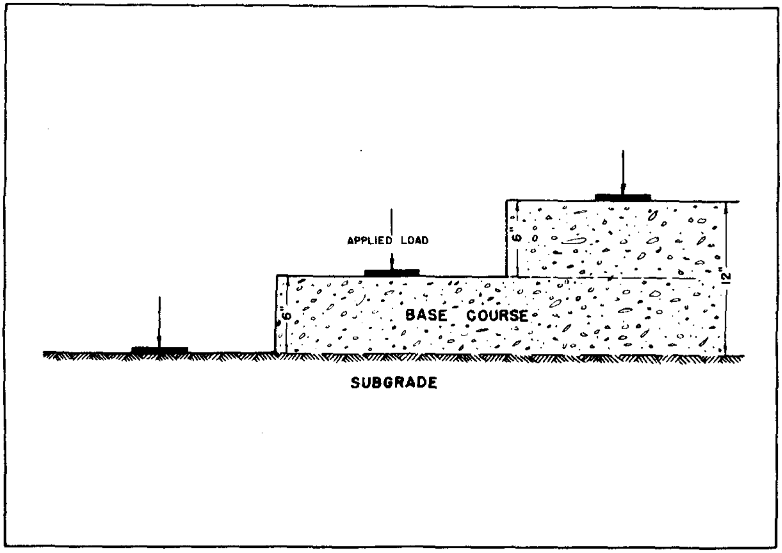


FIG.60 SKETCH OF APPLIED LOADS ON SUCCESSIVE BASE COURSE LAYERS OF UNIFORM THICKNESS

Fig. 61 has been prepared to develop this principle into a simple mathematical equation.

The diagram of Fig. 61 has been drawn on the basis of three assumptions, of which the first two are the two conclusions to be drawn from the data of Fig. 57 that have already been pointed out:

(1) A given thickness of base course has an increasingly greater load carrying capacity when placed on successively stronger subgrades, and vice versa.

(2) The increasingly greater load carrying capacity of a given thickness of base course, when it is placed on successively stronger subgrades, or vice versa, can be expressed as a linear relationship when applied load on the surface of the base is plotted versus subgrade support, line C of Fig. 57.

The first conclusion above from Fig. 57 implies the assumption that has already been stated with regard to Fig. 60, and which is also required for the preparation of the diagram of Fig. 61, namely:

(3) A layer of given base course of specified thickness, normally has a greater load carrying capacity than the subgrade upon which it is placed. A second layer of base course of the same thickness will therefore have a greater load carrying capacity than the first layer,

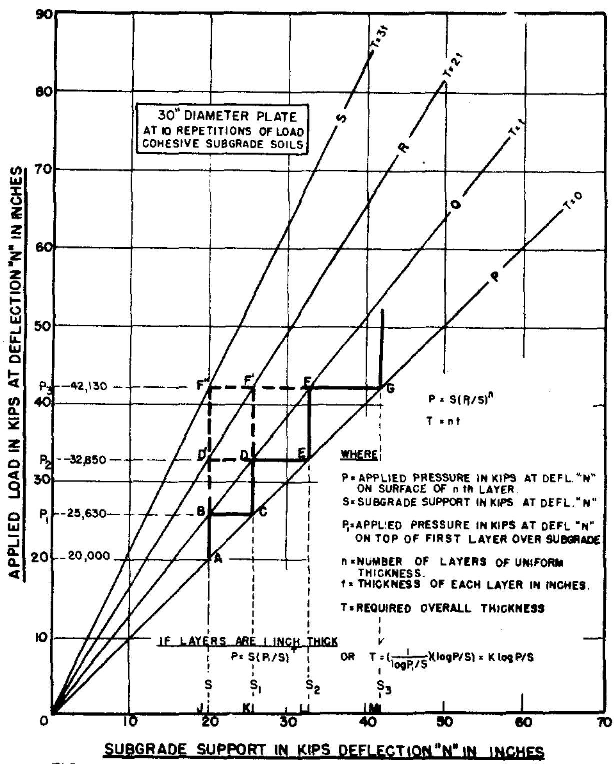


FIG.61 DEVELOPMENT OF DESIGN EQUATION FOR OVERALL THICKNESS

since it rests on a stronger subgrade (the first layer). The third layer has a greater carrying capacity than the second because it in turn rests on a stronger subgrade (the second layer) than the second layer, etc.

It is to be noted that the load supporting capacities of both subgrade and base course refer to the same deflection and same contact area. It is to be also observed that the subgrade support in all cases, is the load indicated by an unconfined load test on the layer in question, and this may be quite different from the actual subgrade support furnished to any layer that is afterwards superimposed. Soil mechanics is unable as yet to provide reliable information on this latter aspect of the problem.

In the diagram of Fig. 61,

(1) Line O P drawn at a slope of 45° gives the value of the subgrade support for any given contact area and any specified deflection N, on either axis.

(2) Point A represents a given magnitude of subgrade support S for a cohesive soil.

(3) Point B represents the applied load P_1 (for the given contact area and deflection N) *carried by a layer of* base course of thickness t, over the subgrade with supporting value S.

(4) Line 0 Q is drawn through point B. B C is drawn horizontally and B A vertically from B.

(5) *Since O P has a slope of* 45°, *it follows from geometry* that A B is equal to B C, and therefore P_1 is equal to S_1. Point C therefore, represents the supporting value, $P_1 = S_1$, of a base course of thickness t.
C D is drawn vertically to 0 Q from C.

(6) The load carrying capacity P_2 of a base course of thickness t placed on subgrade support S_1 (point C), is given by point D.
(From assumption No. 2 above).

(7) Bearing capacity P_2 (point D), is therefore the load carried by a base course of thickness t plus t, or 2t, since point C represents the supporting value P_1 of the first layer of thickness t.

(8) However, the subgrade *support for the base course of* thickness 2t is S, at point A.

(9) D', the intersection of the vertical extension of A B, and of the horizontal through D, therefore represents the value of the applied load P_2, which can be carried by a base course of thickness 2t over subgrade support S.

(10) D E is drawn horizontally through D.
From geometry, A D' is equal to D' E, and P_2 is therefore equal to S_2, point E.
Consequently, point E represents the supporting value, $P_2 = S_2$, of a base course of thickness 2t.
E F is drawn vertically to 0 Q from E.

(11) The bearing capacity P_3 of a base course of thickness t placed on subgrade support S_2 (point *E*), *is given by* point F. (From assumption No. 2 above).

(12) The bearing capacity P_3, (point F), is therefore the load carried by a base course of thickness t + 2t, or 3t, since point E represents the supporting value P_2 of a layer of base *course of thickness* 2t.

(13) However, the subgrade support for the base course of thickness 3t is S, at point A.

(14) F", the intersection of the vertical extension of A B, and of the horizontal through F therefore, represents *the value of the* applied load P_3, which can be carried by a base course of thickness 3t when placed on subgrade support S.

(15) Incidentally, F' represents in turn, the load P_3 which could be carried by a base course of thickness 2t, if the subgrade support were S_1.

(16) Lines O R and O S have been drawn through points D' and F', and through F", respectively.

(17) The above procedure can be followed to determine the bearing value, P_n, of n layers of base course, each of thickness t, over subgrade support S.

From the geometry of similar triangles in Fig. 61,

$$\frac{B\,J}{A\,J} = \frac{D\,K}{CK} = \frac{F\,L}{E\,L} - - - - - - \text{etc.} \tag{1}$$

That is, from what has gone before,

$$\frac{P_1}{S} = \frac{P_2}{S_1} = \frac{P_3}{S_2} = - - - - - - \frac{P_n}{S_n-1} = \frac{P}{S_n-1} \tag{2}$$

where,

- S - subgrade support of original subgrade.
- S_1 - subgrade support given by first layer of base course of thickness t, when placed on subgrade support S.
- S_2 - subgrade support given by first two layers of base course each of thickness t, placed on subgrade support S.
- etc.
- P_1 - load carrying capacity of first layer of base course of thickness t when placed on subgrade support S.
- P_2 - load carrying capacity of first two layers of base course of overall thickness 2t, when placed on subgrade support S.
- $P_n = P$ - load carrying capacity of base course of n layers each of thickness t, when placed on subgrade support S.

but from what has gone before,

$$P_1 = S_1,\ P_2 = S_2,\ P_3 = S_3,\ \text{etc.} \tag{3}$$

therefore, substituting in (2)

$$\frac{P_1}{S} = \frac{P_2}{P_1} = \frac{P_3}{P_2} = - - - - - - \frac{P_n}{P_n-1} = \frac{P}{P_n-1} \tag{4}$$

from which

$$P_2 = \frac{P_1^{\,2}}{S} \tag{5}$$

And it follows from substituting (5) in (4), that

$$\frac{P_3}{P_2} = \frac{P_3}{\frac{P_1^{\,2}}{S}} = \frac{P_1}{S} \tag{6}$$

or

$$P_3 = \left(\frac{P_1}{S}\right)\left(\frac{P_1^{\,2}}{S}\right) \tag{7}$$

Dividing through by S gives

$$\frac{P_3}{S} = \left(\frac{P_1}{S}\right)\left(\frac{P_1}{S}\right)^2 = \left(\frac{P_1}{S}\right)^3 \tag{8}$$

It can be similarly shown that

$$\frac{P_n}{S} = \left(\frac{P_1}{S}\right)^n \tag{9}$$

or

$$\frac{P}{S} = \left(\frac{P_1}{S}\right)^n \tag{10}$$

where $P = P_n$, the load carrying capacity of n layers of base course each of thickness t, when placed on a cohesive subgrade of supporting value S.

It follows that the overall thickness of base course T required to carry applied load P over subgrade support S is,

$$T = n\ t \tag{11}$$

where n = number of layers each of thickness t.

If the layers of base course are considered to be 1-inch thick, equation (10) becomes,

$$\frac{P}{S} = \left(\frac{P_1}{S}\right)^T \tag{12}$$

or

$$T = \left(\frac{1}{\log\ (P_1/S}\right)\ (\log\ (P/S)\ ) \tag{13}$$

Since P_1 is the load carried by a base course of unit thickness when placed upon a given subgrade support S, its value could be expected to vary with the composition, moisture content, and density, of the base course material. It was pointed out in Section 9 however, that for similar conditions of moisture content and density, there is no definite evidence that any one type of granular base has a greater supporting value per inch of thickness than any other type. If it is assumed therefore, that base courses are placed and function under similar conditions of compaction, the expression log (P_1/S) could be considered a constant, and equation (13) would become,

$$T = K \log P/S \tag{14}$$

where

$$K = 1/\log (P_1/S).$$

A further discussion of the value of the expression $1/\log (P_1/S)$ from equation (13), is given in Section 27 below.

21. <u>Evaluating Constant K in Design Equation (14)</u>.

Before equation (14) can be utilized for design, an *average value, or a series of values must be determined for* the constant K. Figs. 62, 63, and 64 have been prepared for this purpose. The data of these three figures all pertain to 0.5 inch deflection, 30-inch diameter bearing plate, and cohesive subgrade soils.

In Fig. 62, total applied load is plotted versus subgrade support for base courses 7 inches thick. The actual thickness in each case varied from about 5 to 9 inches, but the data were corrected on the basis of simple proportion to apply to a thickness of 7 inches.

In Fig. 63 information similar to that of Fig. 62 is given for base courses of 14 inches in thickness. For this diagram, the actual thickness of base course in the field varied from 12 to 16 inches, but the load test data have been corrected on a proportionate basis to apply to a thickness of exactly 14 inches.

A base course thickness of 21 inches has rarely been employed for airport runway construction in Canada, and it was necessary to obtain the information for Fig. 64 somewhat indirectly, on the basis of surface load tests for all test locations where the overall thickness of base and wearing course was in the vicinity of 18 inches. The thickness of the bituminous wearing surface at these test locations was usually from 5 to 6 inches. It was pointed out previously in Section 10 of this paper, that one inch of bituminous pavement containing liquid asphalt or soft asphalt cement binder, was found to have the same load supporting capacity as about 1-1/2 inches of granular base. Applying this principle to the load test data for 18 inches of pavement and base, gave the load test data in terms of granular base 21 inches in thickness, which appear in Fig. 64.

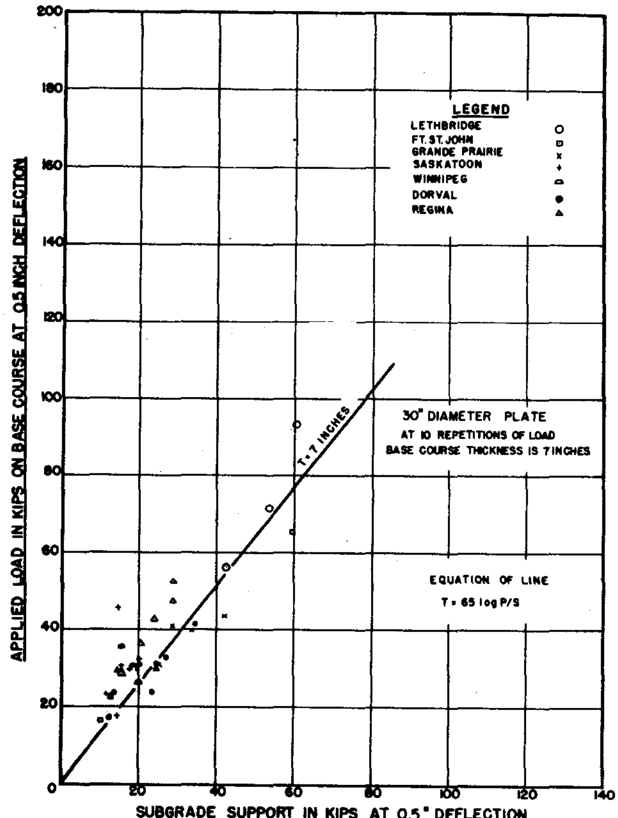


FIG. 6.2 APPLIED LOAD IN KIPS ON BASE COURSE AT 0.5" DEFLECTION VERSUS SUBGRADE SUPPORT IN KIPS AT 0.5" DEFLECTION

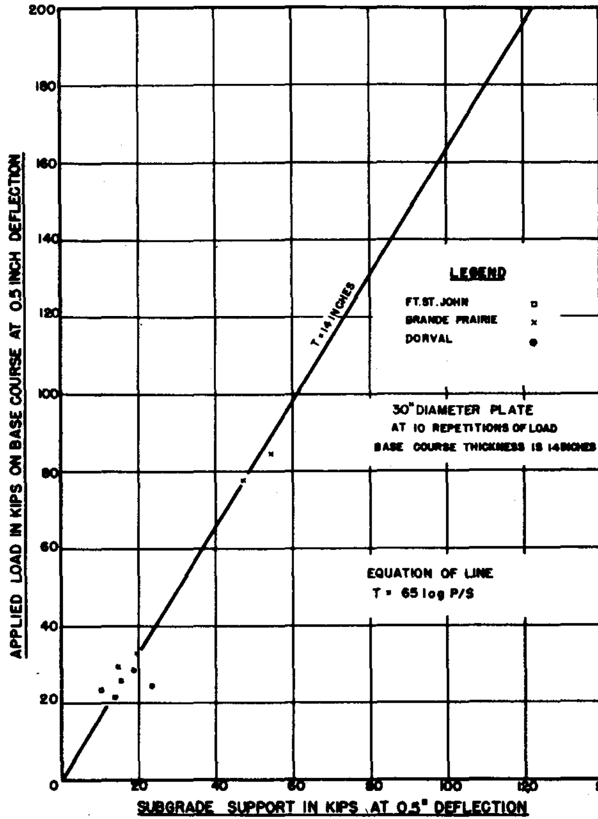


FIG. 6.3 APPLIED LOAD IN KIPS ON BASE COURSE AT 0.5" DEFLECTION VERSUS SUBGRADE SUPPORT IN KIPS AT 0.5" DEFLECTION.

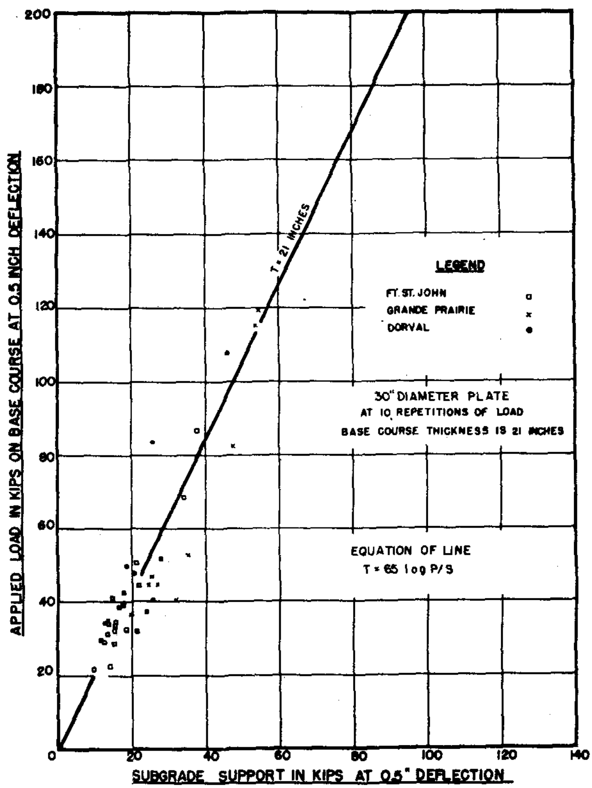


FIG.64 APPLIED LOAD IN KIPS ON BASE COURSE AT 0.5" DEFLECTION VERSUS SUBGRADE SUPPORT IN KIPS AT 0.5" DEFLECTION

The straight line relationships through the data of Figs. 62, 63 and 64, were established by means of trial and error, and represent a value of K = 65. It will be observed that they fit the data very well. In each case therefore, the straight line relationships through the points of Figs. 62, 63 and 64, represent the design equation,

$$T = 65 \log (P/S) \qquad (15)$$

It is to be observed that equation (15) represents the particular case of equation (14) in which the value of K = 65.

22. Design Curves for Thickness of Flexible Pavements for Runways.

The thickness requirements of equation (14) have been indicated graphically in Fig. 65, to illustrate the influence

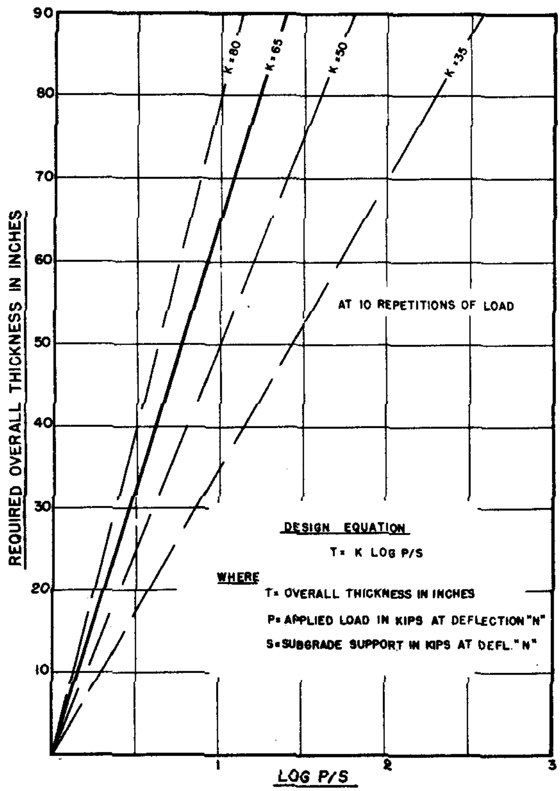


FIG.65 GRAPH OF OVERALL THICKNESS VERSUS LOG P/S

on required thickness of different values of K. For base courses constructed at Canadian airports up to the present time, the best average value of K for design appears to be 65, Figs. 62, 63, and 64. This is indicated by the continuous line labelled K = 65 in Fig. 65. The influence of three other values of K on the required thickness of granular base, has been shown by means of the broken lines in Fig. 65.

While Fig. 65 indicates the required thickness of granular base for any combination of applied load and subgrade support by means of a very simple graph, this information can be expressed somewhat differently through the use of other graphs. Fig. 66 consists of a chart of curves showing the required thicknesses of granular base versus subgrade support at 0.5 inch deflection for a wide range of aeroplane wheel loads for airport runways.

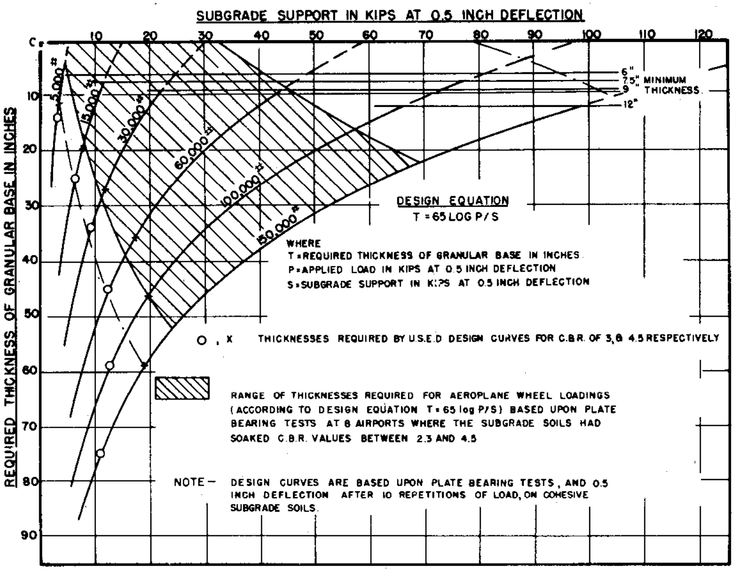


FIG.66 DESIGN CURVES FOR FLEXIBLE PAVEMENTS FOR AEROPLANE WHEEL LOADING (FULL LOAD ON SINGLE TIRE)

The thickness requirements of U.S.E.D. design curves for different aeroplane wheel loadings over subgrades with soaked C.B.R. ratings of 3 and 4.5, are indicated on the curves of Fig. 66, by means of circles and crosses, respectively. The cross-hatched portion of Fig. 66 gives the range of thicknesses indicated by design equation (15) for 8 Canadian airports at which the average soaked C.B.R. ratings of the subgrades varied from 2.3 to 4.5. It should be recalled that the designs represented by the curves of Fig. 66 are based upon 10 repetitions of load, and a subgrade deflection of *0.5 inch.*

It will be noted that the thickness requirements for runways indicated by load tests made at Canadian airports, and based upon observed traffic performance over a period of several years, are materially less than those specified by U.S.E.D. design. Quantitatively speaking, the combined load test and traffic information obtained during this investigation shown that runways with thicknesses of base and surface varying from about one-third to about four-fifths (depending upon climate, depth to water table, etc.) of those required by U.S.E.D. design, have been functioning satisfactorily in Canada.

The broken lines on the extreme left and extreme right of Fig. 66, indicate the thicknesses that would be required on the basis of the lowest and highest subgrade plate bearing values respectively, that were found for the eight airports with cohesive subgrade soils. These values were too far out of line with the others, to be included when evaluating the subgrade support for the airports at which they occurred.

In Fig. 67, the thickness design curves of Fig. 66 have been drawn on the basis of the four tests, cone bearing, Housel penetrometer, field C.B.R., and triaxial compression. Fig. 67 represents the combined information of Figs. 16, 42 and 66. Fig. 67 makes it possible to design the thickness of base course and surface required for runways, upon the basis of the rating of the subgrade as measured by one or more of these simple tests.

23. Thickness Design Curves for Taxiways, Aprons and Turnarounds.

The thickness design curves of Figs. 66 and 67 can be employed to determine the required thickness of base course and surface for any aeroplane wheel loading for runways, except for the turnaround areas at each end, where the

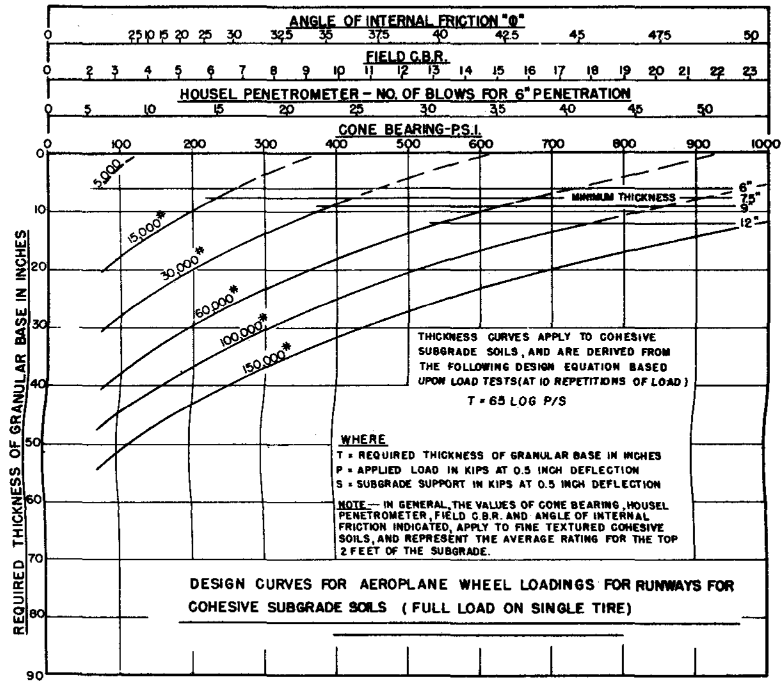


FIG. 67

aircraft turn and pause before take-off. They are based upon 0.5 inch deflection and 10 repetitions of load as determined by plate bearing tests, since these criteria seem to fit in very closely with the performance under traffic of the various runways tested.

When these airports were constructed, the taxiways and aprons were of the same thickness and general construction as the runways. In a large number of cases, the taxiways and particularly the aprons, showed signs of distress under aircraft traffic, after a time. The taxiways were strengthened by the use of a greater thickness of base or by employing rigid pavements, and many of the aprons when reconstructed were provided with a rigid pavement. In some cases, either during construction or afterward, the runways were provided with rigid pavements for the turnaround areas at each end of the runway.

It is obvious, therefore, that the thicknesses of base and flexible surface which are satisfactory for runways, are usually inadequate for taxiways, aprons and turnarounds. Published reports indicate that this has been widely observed elsewhere.

The problem which presents itself therefore, is how to arrive at the greater thickness of base and flexible surface required for taxiways, aprons and turnarounds, as compared with the thickness established for the runways at any airport site.

It is believed that Figs. 21 and 22 provide a rational approach to this problem.

It seems reasonable to expect that the wheels of a standing aircraft will settle steadily into a runway if the yield point of the overall structure, and more particularly if the yield point of the subgrade, is exceeded.

Figs. 21 and 22 indicate that the average yield points of the overall runway structures and of the underlying subgrades, occur at 0.225 and 0.26 inch, respectively. Since these two values check each other so nearly, it is believed that they both represent the yield point of the subgrade. The actual yield point therefore, might be conservatively taken as the lower of these two values, or 0.225 inch.

Consequently, if runway design for flexible pavements is to be based upon the subgrade supporting value for 10 repetitions of load at <u>0.5 inch deflection</u>, since this ties in with Canadian traffic experience, it seems reasonable to base the thickness design for taxiways, aprons, and turnarounds, on the supporting value of the subgrade for 10 repetitions of load <u>at 0.225 inch deflection</u>, the average yield point deflection of the pavement.

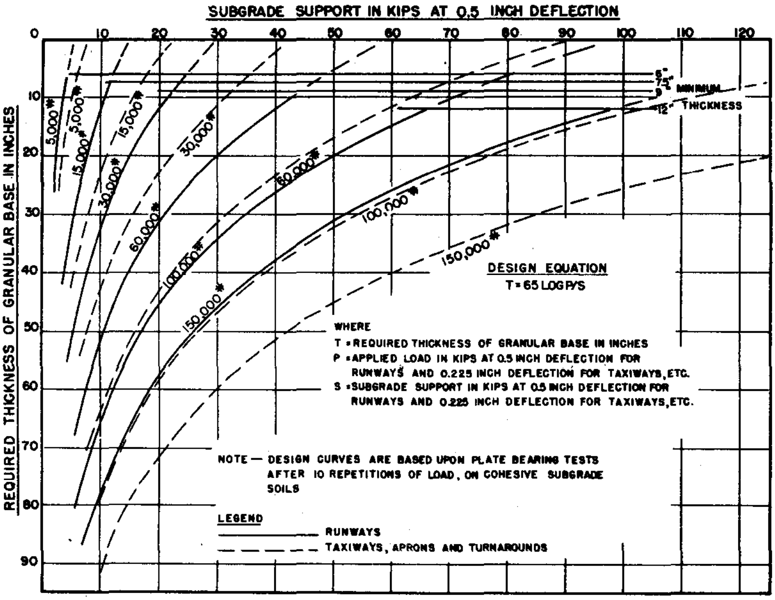


FIG. 68 DESIGN CURVES FOR FLEXIBLE PAVEMENTS FOR RUNWAYS AND TAXIWAYS, ETC. FOR AEROPLANE WHEEL LOADINGS (FULL LOAD ON SINGLE TIRE)

Fig. 68 gives the thickness design curves for a number of aeroplane wheel loadings for runways, and for taxiways, aprons and turnarounds, respectively, based upon plate bearing tests. The curves for runway design are identical with those of Fig. 66, and are based upon the design equation $T = 65 \log (P/S)$ for 0.5 inch deflection. The curves for taxiway, apron, and turnaround design are those given by this same design equation but based upon 0.225 inch deflection. Both sets of curves can be plotted on the same diagram, Fig. 68, since there is a constant ratio between subgrade support at 0.5 inch and at 0.225 inch deflection for each individual size of bearing plate, Fig. 16.

Fig. 69 also gives thickness design curves for runways, and for taxiways, aprons and turnarounds, but in terms of the four tests, cone bearing, field C.B.R., Housel Penetrometer, and triaxial compression. The curves for runway design are identical with those of Fig. 67. Those for taxiway, apron, and turnaround design, are based upon a deflection of 0.225 inch, and have been derived by reference to Figs. 16, 42, and 68.

It is to be observed that the thickness design curves of Figs. 68 and 69 refer to the thicknesses of granular base

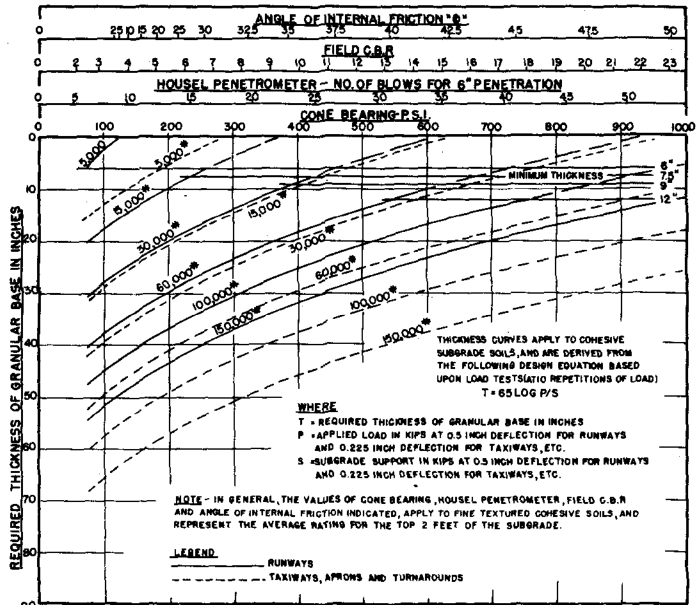


FIG. 69 DESIGN CURVES FOR FLEXIBLE PAVEMENTS FOR RUNWAYS AND TAXIWAYS, ETC. FOR AEROPLANE WHEEL LOADINGS (FULL LOAD ON SINGLE TIRE)

required. It was pointed out in Section 10 above however, that 1-inch of a bituminous pavement containing a liquid asphalt or soft asphalt cement binder, appears to have the load supporting capacity of about 1-1/2 inches of granular base, and that this ratio seems to be about 2-1/2 for well designed and constructed asphaltic concrete, penetration macadam, or sheet asphalt. The maximum thicknesses of bituminous surface to which these ratios can apply, should be taken as 4 inches for the former type, and 6 inches for the latter, until more test data affirms that they may be applied to greater thicknesses.

Some designers may prefer to make no reduction in overall thickness because of the greater load supporting value of the bituminous surface, but to utilize its larger carrying capacity as a safety factor.

By making load tests on existing runways, or taxiways, etc., surfaced with flexible pavements, it is believed that the additional thickness of base and surface required for a heavier wheel loading can be obtained directly from Fig. 68, by assuming that the existing surface will be the subgrade for the new base and surface.

24. Influence of Dual Tires on Flexible Pavement Design for Aircraft.

For a given wheel load, it appears reasonable to expect that the deflection of the pavement will be less if the load is carried on dual tires than on a single tire.

To investigate the quantitative value of dual versus single wheels with regard to runway design, a number of load tests were made with dual and single elliptical steel bearing plates having the same total contact area, Fig. 70. The spacing between the centre lines of the dual plates, 30.75 in., is identical with that for the dual wheels on D.C.4 aeroplanes used by Trans-Canada Air Lines. The dual bearing plates were rigidly yoked together so as to function as a unit in a load test.

The results of a limited number of tests are summarized in Table 7.

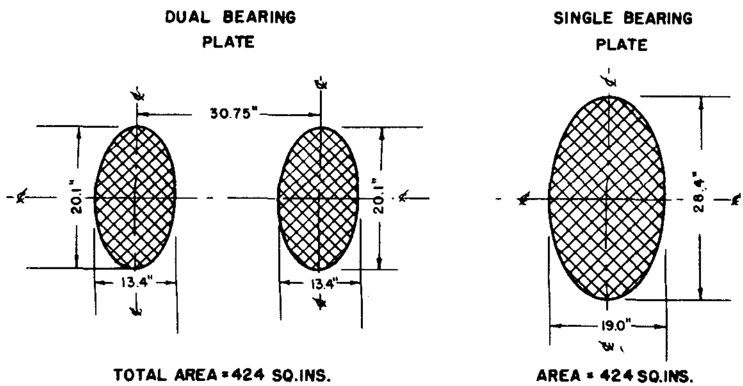


DIAGRAM OF ELLIPTICAL STEEL PLATES EMPLOYED FOR COMPARISON OF SINGLE VERSUS DUAL BEARING PLATE TESTS.

FIG.70

Table 7

Load Supported by Dual versus Single Elliptical Bearing Plates of the Same Total Contact Area, for any given Deflection from 0.2 to 0.5 inch, after 10 Repetitions of Loading.

Airport	Overall Thickness Surface and Base Course Inches	Ratios of $\frac{\text{Load Supported on Duals}}{\text{Load Supported on Single}}$ at any given deflection, for deflection range 0.2 to 0.5 inch.
No. 1	7 to 8	1.35
No. 2	15 to 16	1.25

According to this series of tests, dual wheels of this size and spacing carry from 25 to 35 per cent more load than a single wheel of the same contact area at any given deflection over a range of 0.2 to 0.5 inch. The difference in load carrying capacity of single and dual wheels appears to decrease as the total thickness of base course and wearing surface increased, as would be expected.

The above results are at considerable variance with those reported by the U.S. Corps of Engineers from their studies of the influence of dual versus single tires on runway design for a wheel load of 60,000 pounds for the B-29 Superfortress. They report[6] that for thicknesses of base course and surface up to about 10 inches, this wheel load of 60,000 pounds, if carried on dual tires, had the same effect on runway design as a wheel load of 30,000 pounds on a single tire. That is, for base and surface thicknesses up to 10 inches, the use of a single tire under the given conditions would increase the design load on the runway by 100 per cent above that for dual tires.

Department of Transport data on the other hand, indicate for similar conditions that the use of a single tire increases the design load by only 35 per cent (maximum) above that for duals.

25. Flexible Pavement Design for Highways.

When equation (15), $T = 65 \log (P/S)$, is employed for establishing the thicknesses of flexible base and surface required for various highway wheel loadings over different magnitudes of subgrade support, the design curves of Fig. 71 are obtained.

While the small circles and crosses on the four curves were taken from the U.S.E.D. design chart for aeroplane wheel loadings for subgrades with soaked C.B.R. ratings of 3 and 4.5 respectively, the basis of the U.S.E.D. chart consisted of the results of actual observations of flexible pavement performance in the field made by the California Division of Highways. The thicknesses represented by the circles and crosses were determined by a field survey of the condition of flexible pavements on California highways over subgrades which had field C.B.R. ratings of 3 and 4.5 respectively, *if not permanently, than at sometime during the* year. In the California highway survey, thicknesses of base and surface less than those shown by the circles and crosses, were found to result in failures over subgrades with C.B.R. values of 3 and 4.5 respectively, while greater thicknesses did not.

The shaded portion of the chart of Fig. 71 represents the range of thicknesses for the four highway loadings, which are indicated by design equation (15), when based upon plate bearing tests on subgrades at the eight airports where the subgrade soils had soaked C.B.R. ratings between 2.3 and 4.5. It is to be noted that the maximum thicknesses given by the shaded portion, are in approximate agreement with the thicknesses indicated by California experience for a subgrade with a C.B.R. of 3, for wheel loads of 12,000, 9,000 and 7,000 pounds, but requires about 6 inches less for the wheel load of 4,000 pounds.

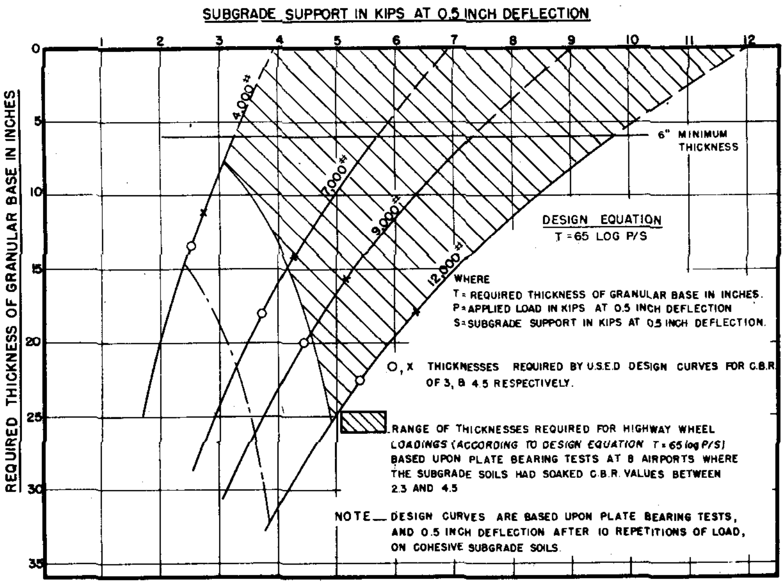


FIG. 71 DESIGN CURVES FOR FLEXIBLE PAVEMENTS FOR HIGHWAY WHEEL LOADINGS (FULL LOAD ON SINGLE TIRE)

It is of considerable interest that a design equation based upon plate bearing tests at 10 repetitions of load, and 0.5 inch deflection, should have indicated maximum thicknesses of pavement which conform so nearly to the maximum depths of base and surface found necessary by an actual survey of flexible pavement performance under traffic in California. Insofar as highway wheel loadings of 9,000 and 12,000 pounds are concerned therefore, it is obvious that if anything, the thickness requirements indicated by design equation (15), $T = 65 \log (P/S)$, are somewhat conservative, rather than otherwise.

It is to be noted that for subgrade soils with soaked C.B.R. values of 3, the four curves of Fig. 71 would contract to the points indicated by the small circle on each curve, if the design recommended by the U.S.E.D. or California Division of Highways were followed, since their designs are based upon the assumption of completely saturated subgrade conditions. It is precisely because all subgrades of fine textured cohesive soils do not become saturated, that the greater flexibility of design indicated by the full curves of Fig. 71, must be considered.

The distance between the circle and cross along each curve of Fig. 71, represents the range of thicknesses permitted by U.S.E.D. and California designs for subgrades with soaked C.B.R. ratings of 3 to 4.5. Actual plate bearing tests on Canadian airports with subgrades having soaked C.B.R. ratings between 2.3 and 4.5, have warranted the very much wider range of thicknesses shown by the shaded portion of Fig. 71. This much wider range of thicknesses is justified because the subgrade soils were saturated at only a small percentage of the test locations.

The shaded area indicates that even for a soaked C.B.R. rating of not over 4.5 the subgrade soil itself had sufficient supporting value in some cases to carry highway wheel loads as high as 12,000 pounds without pavement, although in actual practice a pavement is always required to provide the necessary resistance to the abrasion of traffic, and to protect the subgrade from rain and other weathering agencies.

The broken line on the extreme left of Fig. 71 indicates the thicknesses that would be required for the four highway wheel loadings, on the basis of the lowest subgrade plate bearing value found at any test location. This low value was too far out of line with the others, to be included when evaluating the subgrade support for the airport at which it was found.

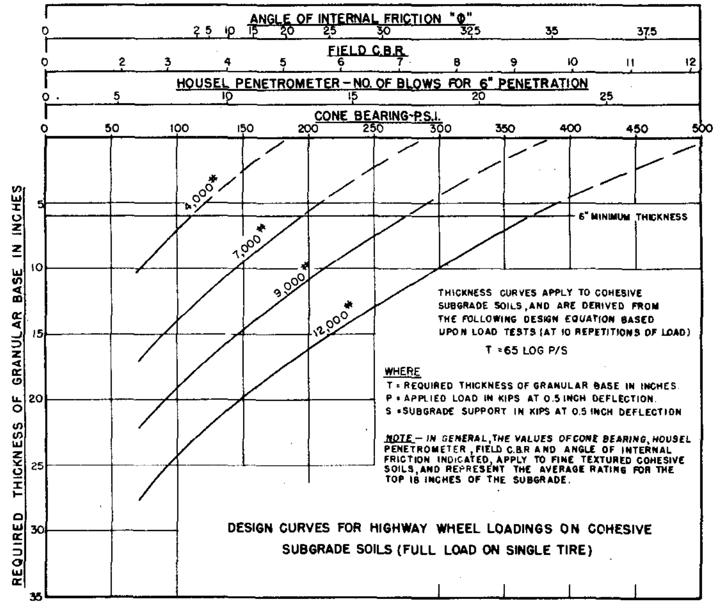


FIG. 72

Fig. 72 provides a chart of thickness design curves for the four highway wheel loadings, based upon cone bearing, Housel penetrometer, field C.B.R., and triaxial compression tests. Fig. 72 was derived by combining the information of Figs. 16, 42, and 71.

It will be observed from Figs. 71 and 72, that the thickness requirements are given in terms of granular base. This thickness can be somewhat decreased on the basis of the type of bituminous surface employed. For bituminous surfaces constructed with liquid asphalt or soft asphalt cement binders, 1 inch of surface appears to have the supporting value of about 1-1/2 inches of granular base, while 1 inch of properly designed and constructed asphaltic concrete, penetration macadam, and sheet asphalt, seems to have the supporting value of about 2-1/2 inches of granular base. The maximum thickness of bituminous surface to which these ratios can apply, should be taken as 4 inches for the former type, and as 6 inches for the latter, until more test data for greater thicknesses become available. However, the designing engineer may not care to make any reduction in overall thickness from that indicated by Figs. 71 and 72, but may prefer to utilize the greater supporting capacity of the bituminous pavement as a safety factor.

26. General Discussion of Flexible Pavement Design Equation (14).

There are two comments with regard to equation (14) for required thicknesses of flexible pavements, that was developed earlier, which should be made at this point.

(1) While the thickness design requirements of Figs. 66, 67, 68, 69, 71, and 72, are all based upon a deflection of 0.5 inch deflection (except the curves for the design of taxiways, aprons, and turnarounds of Figs. 68 and 69), it is to be noted that the fundamental development of design equation (14),

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

does not restrict its use to 1/2 inch deflection. It happens that thickness requirements based upon applied load P and subgrade support S at 0.5 inch deflection and 10 repetitions of load, seem to conform very closely with actual runway service performance under traffic at Canadian airports so far. If at any time in the future, values of P and S at 0.5 inch deflection appear to provide either too great or not sufficient thickness for flexible pavements for runway or highway design, values of P and S at a larger or smaller deflection than 1/2 inch, which would result in smaller or greater thicknesses respectively, can be employed.

(2) The thickness requirements for aprons, taxiways, and turnarounds, given by Figs. 68 and 69, are based upon values of P and S at 0.225 inch deflection. If experience indicates that these thicknesses are either too great or not large enough, values of P and S at a respectively greater or smaller deflection than 0.225 inch, can be selected for equation (14).

27. A General Equation for Flexible Pavement Design.

1. Equation (14)

$$T = K \log (P/S) \tag{14}$$

was developed upon the basis that a layer of given base course of specified thickness will develop successively increasing supporting value as it is placed upon successively greater depths of the same base course material over a given subgrade, the load carrying capacity in each case being equivalent to that which would occur if the layer of base course were placed upon subgrades of cohesive soils having the same supporting values as those measured at the top of different depths of base course, Fig. 61.

A moment's consideration indicates that this continuous increase in supporting value of a given layer of base course when placed upon successively greater depths of base course cannot go on indefinitely. For example, a layer of gravel 1-foot thick would probably add very little to the supporting value of a gravel deposit 100-feet deep, assuming that layer and deposit were alike in every respect. Consequently, the graph of Fig. 65, instead of being a straight line, should probably be a curve which is generally concave upwards as demonstrated by the broken line curve of Fig. 73. That is, the value of K should not be a constant which is independent of the depth of any given base course, as indicated in Fig. 65, but should vary with the depth of base course required.

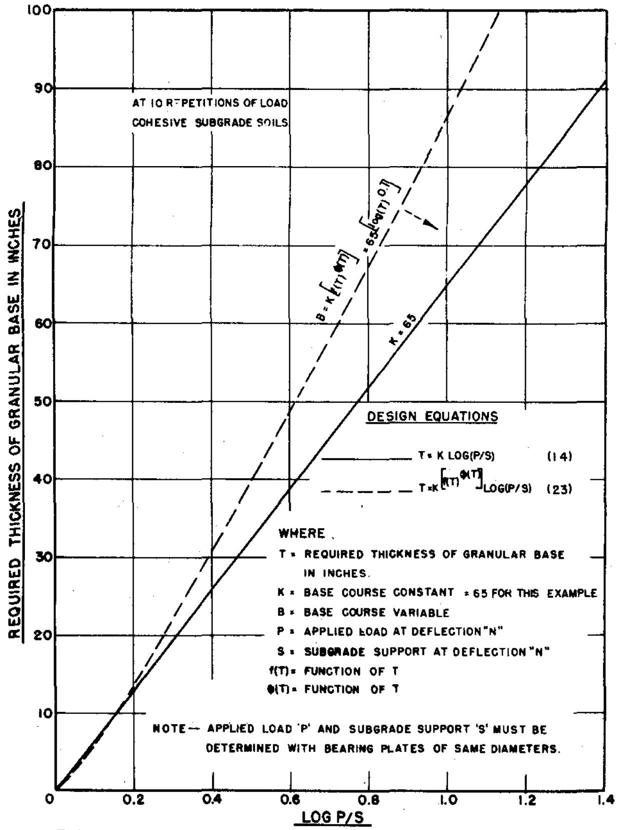


FIG. 73 INFLUENCE OF THE NATURE OF THE BASE COURSE VARIABLE ON REQUIRED THICKNESS FOR FLEXIBLE PAVEMENT DESIGN.

It is necessary, therefore, to reconsider equation (14)

$$T = K \log (P/S) \tag{14}$$

to learn what modifications are required in order that a more general equation for flexible pavement design may be developed.

Examination of the right hand side of equation (14) indicates that it consists of parts I and II as shown below

$$T = \underbrace{(K)}_{\text{I}} \qquad \underbrace{(\log (P/S))}_{\text{II}}$$

Part II, the expression log (P/S), seems to be independent of the depth of base course and is therefor valid as it stands. Geometrically it follows from Fig. 61, and from the development of equations (4) to (14) in Section 20 above, that the expression log (P/S) is independent of the thickness of base course. For example, the geometrical arrangement of steps in Fig. 61 for a base course layer of thickness t, from which the expression log (P/S) was derived, holds for a base course of any given depth from a fraction to a multiple of t, since the relation between surface load versus subgrade support for a base course of any thickness, is expressed by a straight line through the origin, e.g., Figs. 62, 63 and 64.

The expression log (P/S) of equation (14) therefore, results from the straight line relationship when applied load P is plotted versus subgrade support S for a base course of any given thickness, which is one of the fundamental conclusions indicated by the Department of Transport's investigation. Unless this fundamental conclusion is found to require modification as a result of further study, this portion of equation (14) is valid as it stands.

Part I, or K, of equation (14), has been considered thus far as a constant which has an average value of 65, based upon load test data from the Canadian airports so far investigated. The value of K for the base course at Regina Airport, however, was about 35. Consequently it appears that the value of K for any particular base course in place may depend upon the composition, moisture content, and density of the base course material.

From equations (13) and (14), it will be seen that $K = 1/\log (P_1/S)$. A review of Section 20 indicates that the expression $1/\log (P_1/S)$ may very well be a variable, since P_1 is the load supported by a unit depth of base course over subgrade support S. It is clearly quite probable that the value of P_1 may depend upon the composition, moisture content, and density of the base course material.

In the opening paragraph of this section it was pointed out that the value of the right-hand side of equation (14) could be expected to vary with the depth of base course. It has already been indicated that Part II of the expression on the right, i.e., log (P/S), appears to be independent of the depth of the base course. Therefore, it must be Part I, or the value of the expression K of equation (14) which varies with the depth of base course.

Consequently, for the general case, the value of K of equation (14) may vary not only with differences in the composition, moisture content, and density of the base course material from project to project, but may also vary with the depth of any given base course even when all other factors are kept constant.

From these various considerations, it appears that for the general case, the required thickness of base course would be given by the following expression:

$$T = (1/\log\ (P_1/S))^{f(T)} \log\ (P/S) \tag{16}$$

where T = required thickness of granular base in inches.

P_1 = load supported at a given deflection by a unit thickness of any given base course on subgrade support S.

P = applied load at given deflection.

S = subgrade support at given deflection.

$f(T)$ = function of thickness T.

It is more convenient to write equation (16) as

$$T = K^{f(T)} \log\ (P/S) \tag{17}$$

in which

$$K = 1/\log\ (P_1/S) \tag{18}$$

where K = base course constant, and for any given base course it has the value given by the expression $1/\log\ (P_1/S)$. The value of the base course constant K depends therefore, upon the composition, moisture content, and density of a unit thickness of each base course material in place.

The exponential term, f(T), appearing in equation (17), indicates that the value of K determined for a unit thickness of any given base course, may be dependent also upon the depth of the base course.

Before equation (17) can be used, it is necessary to be able to evaluate the expression f(T), the function of the thickness T, which appears as an exponential term in this equation.

It is obvious that for relatively small thicknesses, the value of the expression $K^{f(T)}$ of equation (17) will not be greatly different from the value of the expression K of equation (14). Consequently, the following relationship appears to be reasonable:

$$K^{f(T)} = K^{(\log T)^r} \tag{19}$$

in which

$$f(T) = (\log T)^r \tag{20}$$

That is, f(T), the function of the thickness T required, is the logarithm of the thickness T raised to the r^{th} power, where r is a fraction having a value between zero and unity.

There is a possibility that the exponential term "r" of equations (19) and (20) may not be constant, but that it also varies with the thickness T. That is, "r" may also be a function of T, or

$$r = \varphi(T) \tag{21}$$

The general equation for the required thickness of flexible pavements on the basis of this development becomes then,

$$T = K^{\left(f(T)^{\varphi(T)}\right)} \log (P/S) \tag{22}$$

or

$$T = K^{\left((\log T)^{\varphi(T)}\right)} \log (P/S) \tag{23}$$

For equations (22) and (23), the values of K and $\varphi(T)$ must be determined experimentally before they can be employed for actual design.

Equations (22) and (23) can be written in the following simpler form,

$$T = B \log^{(P/S)} \tag{24}$$

in which

$$B = K^{\left((\log T)^{\varphi(T)}\right)} \tag{25}$$

where B = the base course variable for a unit thickness of any given base course material in place. Equations (18) and (25) indicate that the value of B depends upon the composition, moisture content, density, and depth of the base course material in place on any project.

The influence of thickness T on the value of the expression $B = K^{\left((\log T)^{\varphi(T)}\right)}$ is shown in Table 8 for a wide range of values of $\varphi(T)$, when the value of K is taken as 65 by way of example.

From the data of Table 8, it is apparent that values of $B = K^{\left((\log T)^{\varphi(T)}\right)}$ of equations (23) or (25) depart very little from the value of K in equation (14), for small values of $\varphi(T)$. For values of $\varphi(T)$ greater than about 0.1 on the other hand, the value of $B = K^{\left((\log T)^{\varphi(T)}\right)}$ from equations (23) or (25) may deviate widely from the corresponding value of K from equation (14).

Table 8

Influence of the magnitude of $\varphi(T)$ on the values of $B = K^{\left((\log T)^{\varphi(T)}\right)}$ for the thicknesses of T indicated, when the value of K is taken as 65 for the purpose of illustration.

Values of $B = K^{\left((\log T)^{\varphi(T)}\right)}$ for the thicknesses indicated, when $\varphi(T)$ varies from 0 to 1.0.

Thickness T Inches	$\varphi(T)$ Values										
	0	0.02	0.04	0.06	0.08	0.1	0.2	0.4	0.6	0.8	1.0
5	65.	63.1	61.2	59.5	57.8	56.1	47.1	37.2	29.0	23.0	18.5
10	65.	65.	65.	65.	65.	65.	65.	65.	65.	65.	65.
20	65.	66.4	67.9	69.5	71.0	72.6	81.4	103.3	132.7	172.9	228.4
30	65.	67.2	69.4	71.7	74.2	76.7	91.2	131.5	195.4	299.8	476.3
40	65.	67.6	70.4	73.3	76.3	79.5	98.2	154.5	254.3	439.7	802.4
50	65.	67.9	71.1	74.4	77.9	81.6	103.7	174.2	310.2	589.2	1203.
60	65.	68.2	71.6	75.3	79.1	83.2	108.2	191.5	363.7	746.6	1674.
70	65.	68.4	72.1	76.0	80.2	84.6	112.0	207.1	415.1	910.6	2213.
80	65.	68.6	72.5	76.6	81.0	85.8	115.3	221.4	464.7	1080.	2819.
90	65.	68.7	72.8	77.1	81.8	86.8	118.3	234.5	512.8	1255.	3490.
100	65.	68.9	73.1	77.6	82.5	87.7	120.9	246.7	559.6	1434.	4225.0

For each vertical column of data in Table 8, it will be observed that $\varphi(T)$ has a constant value. If $\varphi(T)$ should vary with the thickness T as its symbol permits, the influence of thickness T on the value of $\varphi(T)$ can be inferred by reading the data of Table 8 from top to bottom in a generally diagonal direction.

It should be noted particularly, that when $\varphi(T) = 0$, equations (22) or (23) become identical with equation (14).

An example of the difference in thickness of granular base required by the use of equations (23) and (14) is given in Fig. 73. The straight line is a graph of equation (14), while the curve represents equation (23) for a value of K = 65, and a value of $\varphi(T) = 0.1$. The divergence in thickness requirements given by the two equations is seen to be quite marked, particularly for the greater thickness values. The divergence illustrated by Fig. 73 has been exaggerated however, if the value of $\varphi(T) = 0.1$ employed by way of example, is considerably higher than experimental data would indicate.

Fig. 74 is similar to Fig. 61 which was employed to illustrate the development of equation (14). The full lines OQ, OR, OS, OT, and OU, for thicknesses of 5, 10, 20, 30 and

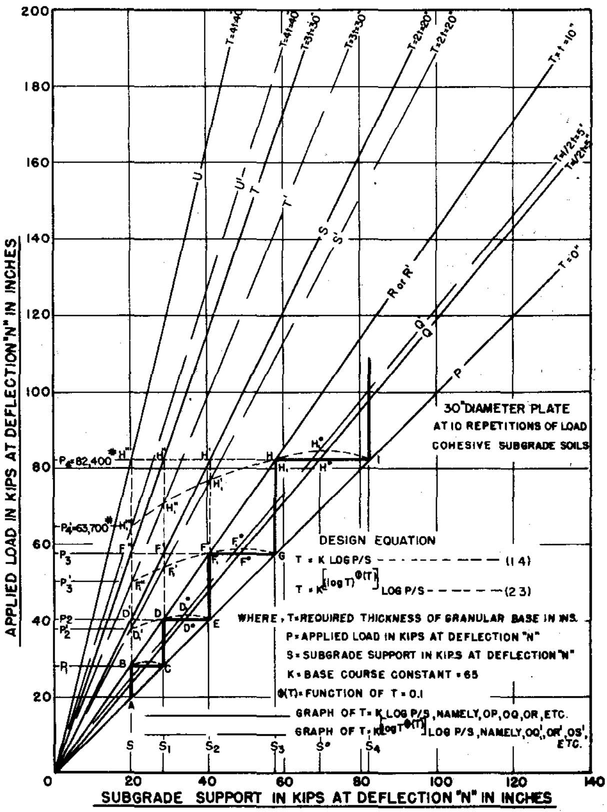


FIG. 74 INFLUENCE OF THE NATURE OF FLEXIBLE PAVEMENT DESIGN EQUATIONS ON APPLIED LOAD VERSUS SUBGRADE SUPPORT.

40 inches of base course, respectively, are drawn on the basis of equation (14), $T = K \log (P/S)$, using a value of $K = 65$. The broken lines OQ', OR', OS', OT', and OU', for base course thicknesses of 5, 10, 20, 30 and 40 inches respectively, pertain to equation (23), $T = K^{\left((\log T)^{\varphi(T)}\right)} \log (P/S)$, using a value of $K = 65$, and a value of $\varphi(T) = 0.1$.

For any given value of thickness greater than 10 inches, it is obvious from Fig. 74, that for any specified subgrade support S equation (14) gives a higher load carrying capacity P than equation (23), since the full lines OS, OT, and OU, cut the ordinate representing any given subgrade support at a higher value of P, than do the broken lines OS', OT', and OU', respectively. For a subgrade support of 20,000 pounds for example, and for a thickness of 40 inches, P_4 given by equation (14) is 82,400 pounds, whereas P_4' given by equation (23) is 63,700 pounds.

The horizontal straight line I, H^o, H, H', H", H'", of Fig. 74, shows the constant value of the applied load P_4, which would be supported by various thicknesses of base course, 0, ½t, t, 2t, 3t, and 4t, over different degrees of subgrade support S_4, S^o, S_3, S_2, S_1, and S, respectively, on the basis that K is a constant which is independent of depth, as indicated by equation (14). The curved line I, H_1^o, H_1, H_1', H_1'', H_1''', on the other hand, demonstrates the variable nature of the applied load which is supported by base course thicknesses of 0, ½t, t, 2t, 3t, and 4t, over the same degrees of subgrade support, S_4, S^o, S_3, S_2, S_1, and S, respectively, when the magnitude of the applied load is calculated by means of equation (23). Similar differences between equations (14) and (23) are indicated by the horizontal straight line G, F^o, F, F', F", versus the curved line G, F_1^o, F_1, F_1', F_1'', and by E, D^o, D, D', versus E, D_1^o, D_1, D_1', etc.

The difference in load carrying values given by equations (14) and (23) for any specified thickness of base course when all other conditions are the same, may be somewhat exaggerated in Fig. 73, since future experimental data might indicate that a value of $\varphi(T) = 0.1$ is too high to use in equation (23).

A careful and comprehensive research program would be required to evaluate the expression $B = K^{\left((\log T)^{\varphi(T)}\right)}$ of equation (23), for the wide range of base course conditions, composition, moisture content, density, and thickness, which probably influence its value.

2. Assuming that values of K and $\varphi(T)$ have been established, the thickness requirement T given by equations (22) or (23) can be determined very easily by a series of successive approximations, as shown below.

1st approximation

$$T_1 = K \log (P/S) \tag{14}$$

2nd approximation

$$T_2 = K\left((\log T_1)^{\varphi(T)}\right) \log (P/S) \tag{23}$$

3rd approximation

$$T_3 = K\left((\log T_2)^{\varphi(T)}\right) \log (P/S) \tag{23}$$

It will be shown by an actual set of calculations, that the 3rd successive approximation carried out as indicated above, will give the actual thickness required within a fraction of an inch. That is,

$$T_3 = T = \text{the required thickness.}$$

3. Sample calculation for obtaining required thickness T by means of equation (23) for an aeroplane wheel loading on the basis of the following data,

Applied load P = 100,000 pounds
Subgrade support S = 20,000 pounds
Base course constant K = 65
$\varphi(T) = 0.06.$

1st approximation

$$T_1 = K \log (P/S)$$
$$T_1 = 65 \log (100{,}000/20{,}000)$$
$$T_1 = 45.43 \text{ inches.}$$

2nd approximation

$$T_2 = K\left((\log T_1)^{\varphi(T)}\right)$$
$$T_2 = 65\left((\log 45.43)^{0.06}\right) \quad \log (100{,}000/20{,}000)$$
$$T_2 = 51.7 \text{ inches.}$$

3rd approximation

$$T_3 = K\left((\log T_2)^{\varphi(T)}\right)$$
$$T_3 = 65\left((\log 51.7)^{0.06}\right) \quad \log (100{,}000/20{,}000)$$
$$T_3 = 52.1 \text{ inches.}$$

Therefore for design $T_3 = T = 52$ inches.

In this example, the required thickness was given within a fraction of an inch by the 2nd approximation.

4. When designing for the thickness of flexible pavements for highways, or for aeroplane wheel loadings for which moderate thicknesses of base course are indicated, the thickness requirement will probably be given with sufficient accuracy by equation (14),

$$T = K \log (P/S) \tag{14}$$

It would seem that only for aeroplane wheel loadings of about 40,000 to 50,000 pounds or more, that are to be carried by runways over low subgrade support, for which considerable thicknesses of base course are needed, and where the use of equation (14) might lead to under-design, would the use of equation (23) or (24) become necessary,

$$T = K\left((\log T)^{\varphi(T)}\right) \log (P/S) \tag{23}$$

$$T = B \log (P/S) \tag{24}$$

5. In equation (23)

$$T = K\left((\log T)^{\varphi(T)}\right) \log (P/S) \tag{23}$$

the right-hand side consists of the three terms which might be expected to enter into flexible pavement design,

(a) The applied load P to be carried.
(b) The subgrade support S that can be mobilized at the deflection specified.
(c) The base course factor $K\left((\log T)^{\varphi(T)}\right)$ which depends upon the composition, moisture content, density, and thickness of the base course material.

6. It was pointed out in item 4 above, that for highway wheel loadings, and for moderate thickness requirements for aeroplane wheel loadings, the required thickness seems to be given with sufficient accuracy by equation (14). Only for the heavier aeroplane wheel loads, which must be carried on runways with low subgrade support, does the use of equation (23) appear to be indicated, since for these cases equation (14) might give thicknesses that are too small, and therefore lead to under-design.

On the other hand, as a result of their investigations, the U.S. Corps of Engineers have suggested that since the radius of curvature increases with the size of the wheel loading (larger imprint area), the allowable flexible pavement deflection to be considered for runway design may be greater for large aeroplane wheel loadings than for smaller wheel loadings[17]

If the allowable deflection of a flexible pavement varies directly as some function of the anticipated wheel load, the curved line graph of equation (23) in Fig. 73 would diverge much less than is shown from the straight line graph of equation (14), since the value of the subgrade support S increases as the permissible deflection is increased, Fig. 16. If the subgrade support S is increased for any given applied load P, the value of the expression log (P/S) becomes smaller. It is generally true that the larger values of log (P/S) apply to the wheel loadings of the heavier aeroplanes. Consequently, if the permissible deflection increases with an increase in wheel load, the top portion of the curved line graph of equation (23) in Fig. 73, would tend to approach toward the straight line graph of equation (14), as indicated by the dotted arrow.

Therefore, if it should be true that the permissible deflection of a flexible pavement can be increased as the wheel load is increased (for heavier wheel loadings), equation (14) (based upon a constant deflection throughout) may have a wider range of application than the previous development in this section would suggest.

7. For graphs of equations (14), (23), etc., which have been shown in a number of diagrams for this paper, a constant deflection for both P and S in the expression log (P/S) has been assumed over the whole range employed in each case. It is to be emphasized however, that these equations are equally applicable if a variable deflection is assumed for different values of P and S (provided that corresponding values of P and S are always taken at the same deflection), although the graphs would have a somewhat modified shape. Consequently, equations (14), (23), etc., will hold, even if it should be adequately demonstrated that the permissible deflection for flexible pavement design is a function of the wheel load or tire imprint area, (greater radius of curvature), as suggested by the investigations of the U.S.E.D.

For the sake of clarity however, it might be preferable to consider that when P and S of the expression log (P/S) appear without subscripts, as in the present notation for equations (14), (23), etc., a constant critical deflection, e.g. 0.5 inch, applies throughout. On the other hand, if the deflection at which the values of P and S are to apply, is to be a variable, e.g., the deflection is to vary as some function of the wheel load, or of the tire imprint area, etc., then both P and S would carry a suitable subscript. The letter "d" is suggested. Under these conditions,

equation (14) would be written as,

$$T = K \log (P_d/S_d) \tag{30}$$

equation (23) would be written as,

$$T = K^{\left((\log T)^{\varphi(T)}\right)} \log (P_d/S_d) \tag{31}$$

It would be understood that while the values of P_d and S_d each depend upon the variable permissible deflection to be employed, the corresponding values of P_d and S_d would always refer to the same deflection, and to the same contact area.

Summary

1. This paper outlines the results of an investigation of runways at a number of Canadian airports which has been conducted during 1945 and 1946 by the Department of Transport.
2. Traffic experience at several of Canada's busier airports, indicates that the current flexible pavement thickness requirements for runways according to the design criteria of several principal organizations in the U.S.A., are ultra conservative.
3. By means of a pedological soil survey, the areas of subgrade with different engineering properties at any airport site, can be ascertained and mapped.
4. Field moisture and density data demonstrate that at only a small percentage of test locations could the subgrade be considered to be saturated.
5. Plate bearing equipment, repetitive load testing procedure, and the method of plotting the load test data are described.
6. Load test data versus traffic information at Canadian airports, indicates that safe runway design can be based upon a deflection of 0.5 inch for ten repetitions of load.
7. For any given deflection for plate bearing tests on cohesive soils, a straight line relationship appears to hold for unit load support versus the P/A ratios of a series of steel bearing plates, over the range of bearing plate diameters between 12 and 42 inches, and probably beyond.

8. It is indicated by means of useful correlations, that if the load supported at 0.2 inch deflection on a 30-inch diameter plate has been accurately determined for a given test location on a cohesive subgrade soil or on a flexible surface, the average load supported at any other deflection between 0.0 and 0.7 inch for bearing plates between 12 to 42 inches in diameter, and probably beyond, may be calculated.
9. The average yield point deflection for subgrades seems to occur at 0.26 inch deflection, and for bituminous surfaces appears to be 0.225 inch, for the airports so far included in the investigation.
10. Base course support per unit of thickness may be generally independent of the composition of granular base course materials, but appears to be influenced by base course density.
11. Bituminous surfaces seem to have a greater load carrying capacity per unit of thickness than do granular bases. The ratio appears to be about 1.5 for bituminous surfaces containing liquid asphalt and soft asphalt cement binders, and about 2.5 for properly designed and constructed asphaltic concrete, penetration macadam, and sheet asphalt.
12. Relationships have been established for plate bearing test results versus cone bearing, Housel penetrometer, field C.B.R., and triaxial compression test data, respectively.
13. A method for designing bituminous paving mixtures by the triaxial compression test is outlined.
14. The use of the triaxial compression test for selecting base course materials of adequate stability is described.
15. Evaluation of the load test data for flexible pavement design, indicates that for any specified deflection, the supporting value of any given thickness of base and surface depends directly upon the magnitude of the subgrade support. This in turn leads to a method of flexible pavement design.
16. Thickness design curves have been prepared to indicate the required thickness of granular base for runways, and for taxiways, aprons, and turnarounds, for a wide range of aeroplane wheel loadings. One set of curves is based upon plate bearing tests, and another set on bone bearing, Housel penetrometer, field C.B.R., and triaxial compression tests.
17. Data are given for dual versus single tires for supporting aeroplane wheel loads.
18. Charts of thickness design curves for flexible pavements for highway wheel loadings have been prepared, based upon

plate bearing tests, and upon cone bearing, Housel penetrometer, field C.B.R., and triaxial compression tests.
19. General equations of design for required thickness of flexible pavements have been developed, based upon subgrade support, base course support per unit thickness of base, and applied wheel load.

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Arrangements were made to have the soil samples from the seven airports investigated in Western Canada, tested at the University of Alberta, under the direction of Dean R. M. Hardy. The soil samples from Dorval airport were tested at McGill University under the supervision of Mr. G. A. Leonards. The testing of soil samples from Uplands (Ottawa), and from Malton (Toronto), was carried out at the University of Toronto under Professor R. F. Legget, assisted by Mr. R. L. Davies.

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The writer desires to express his warm appreciation for the kind assistance and cooperation received from everyone associated with the investigation. We wish to thank P. J. Prokopy for his craftsmanship and patience in drafting the large number of figures and diagrams required. Special mention should be made of the very able assistance provided by C. L. Perkins, J. P. Walsh, R. Applebaum, D. S. Johnson, and B. H. Newington, in working up and correlating the test data.

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Discussion

MR. E. W. BAUMAN (National Slag Association): The findings of the investigation which permitted the conclusions that an inch of bituminous material, using the cut-back or soft asphalt, was equivalent in bearing value to about 1-1/2 inches of granular material - macadam or the like - and that 3 inches of asphaltic concrete was the equivalent to approximately 6 inches of granular construction is interesting. However, I wonder, if because of what you stated later in your discussion, that as you increase the number of layers of the sub-base material, each succeeding layer provides a greater bearing value, what would you get if bearing tests were made by reversing the order of the layers? That is, put the bituminous layer directly over the subgrade with the granular layer on top of it. Wouldn't it be possible that you would find that the layer of the granular material to have at least the same value as the bituminous layer?

MR. McLEOD: While we have no load test information with regard to the exact problem about which you enquire, we would infer that the increase in load supporting capacity obtained from placing a given thickness of granular base on an existing bituminous surface would be no greater than that obtained by placing the same base course on a clay subgrade of the same bearing capacity as the bituminous surface. The increase in bearing capacity provided by the granular base course in this case would be less than that provided by the same thickness of bituminous pavement, provided the range of thickness was not large. This at least is the interpretation we place on our present data.

We would also infer that a bituminous surface of a specified thickness, when placed on a given granular base over a clay subgrade, would increase the load carrying capacity of the structure by a greater amount than an equal thickness of granular base course material, all other conditions being the same. The ratios of the increase in supporting capacity provided by the bituminous surfacing versus the granular base for these conditions, would be those given in the paper, at least as revealed by our own data. It is possible, however, that if granular base courses could be placed at greater relative densities than are obtained during average construction practice at the present time, the quoted ratios might be reduced.

MR. H. G. NEVITT (by letter): This report on Canadian data and its analysis is another helpful contribution by Mr. McLeod, and adds to his record of careful study in the critical problem of flexible pavement design.

Without in any way questioning that the data obtained to a considerable extent substantiates the analysis, I still believe that the application of the relationships worked out to general design purposes lacks justification without much more evidence, since it is subject to serious criticism from theoretical standpoints. Two relationships will be discussed.

The first is that when any bearing value for a given plate diameter is known the value for any other size plate for the particular structure tested can be predicted. This implies that the strength of a soil is a unidimensional quantity, at least under constant conditions of moisture content, compaction, and similar. Incidentally this same assumption is likewise implied in the C.B.R. method. Without getting into the merit of whether the C.B.R. testing procedure accurately predicts the bearing value under the most severe moisture conditions to be expected, or that field bearing tests can be made under conditions which will correspond to this critical moisture situation, the merit of this assumption as to the unidimensionality of soil as a structure is strongly questioned. All our knowledge seems to be to the contrary. Triaxial tests clearly show that soil may have two independent properties (namely, the angle of friction and cohesion) and there is nothing to indicate these may always be interrelated even though this appears to have been approximately the case for the series of air fields tested. Housel's work on bearing tests likewise indicates that soil strength is a two-dimensional quantity, and whether or not it is defined by the two quantities of bearing power and perimeter shear as questioned in one example by Mack or whether some other situation exists, it seems clear that two soils can exist which show the same bearing power for one size plate yet a quite different bearing power for other plates appreciably different in size. Too much data to this effect exists to in the writer's opinion justify any general position that soil strength is a unidimensional quantity and can be determined by a bearing test with one plate, a bearing value with one plunger, or similar.

Another relationship that is theoretically dubious is the formula for pavement thickness to support any load value over a given bearing value soil, when both are measured with the same size plate. This implies that the load bearing capacity for any pavement or base course layer increases exponentially with the thickness, as against an increase in bearing value of elastic materials which display beam action with the square of the thickness. It seems obvious that this relation cannot have beneral applicability because of the tremendous bearing value which would shortly be reached with thicknesses which are still of a reasonable magnitude. It seems just as questionable, even though the exponential curve may coincide with the observed data over the range of thicknesses recorded, that it holds for all structures of this type. As a matter of fact, to this implicit doubt concerning the formula can be added explicit disagreement with its derivation. This essentially depends upon the assumption that any subgrade strengthened by a given thickness of surfacing will act exactly like another subgrade of the same (greater) strength, and therefore require the same further increment in thickness to carry a desired load as would the stronger subgrade. This assumption does not seem justified. Theoretical considerations of the mechanics of soil (or aggregate mass) action indicate the existence of at least two independent properties (1). Furthermore as Burmister (2) has pointed out, the characteristics of a two layer system are quite different than those of a one-layer system; it is consequently not at all justifiable to assume that this incremental layer will give the same support to the weak subgrade plus a foundation course that it would supply to a subgrade of the same strength as the reinforced weaker subgrade.

While these theoretical doubts demand caution in any attempt to generally apply the formulae worked out by Mr. McLeod to design where the conditions are not known to be practically identical with those for which the data was obtained, this does not mean that they may not have great merit. Certainly designs developed in accordance with these procedures should be compared against those arrived at by other methods in order to get a better evaluation of the limits in design arrived at for any proposed structure. Likewise it may be found that the resemblance of so many structures to the Canadian fields tested is sufficiently close that these design procedures may be tentatively justifiable with the corollary conclusion that stronger structures than they demand are not justified unless actual bearing tests or similar clearly show that greater investment is required. In brief, while Mr. McLeod's data seems to cover a series of structures which are similar in nature and therefore the design formulae worked out to suit them may not yet justify general application on theoretical grounds, they still may in practice prove to be quite helpful. However this confirmation must be definitely worked out before we can safely use them without cautions, although at the same time it is certainly obvious that the excessive structures now being called for by some design techniques must clearly demonstrate that they are necessary before sound engineering will permit the commitment to the high construction costs they imply.

The flexible design problem seems to need both data and discussion for its final solution, and this paper of Mr. McLeod's is certainly a contribution from both standpoints.

(1) Nevitt, H. G., "A Mathematical Analysis of Some Phases of the Flexible Surface Design Problem" Highway Research Board Proceedings, 1943, p. 149.

(2) Burmister, Donald M., "The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways," Highway Research Board Proceedings, 1943, p. 126.

MR. McLEOD (by letter): Mr. Nevitt's discussion was written on the basis of the oral presentation of the paper, which had to be relatively brief. Some of his comments have been answered, at least in part, in the paper as published.

From a theoretical standpoint, Mr. Nevitt questions the statement that if the bearing value of a soil has been determined with a given bearing plate, the bearing value can be calculated for a plate of any other size. He states that

"...it seems clear that two soils can exist which show the same bearing power for one size plate yet a quite different bearing power for other plates appreciably different in size."

We have no doubt that two such soils may occur. A clay soil of dense structure, and one of porous flocculated structure, might be examples, because of differences in the shapes of the load versus deflection curves that might be expected for each. However, for the ten airports included so far in the Department of Transport's investigation, where the subgrade soils are representative of those normally occurring in Canada, our load test data made with different sizes of bearing plates, seem to justify the chart of Fig. 19, which indicates that if the load carried by a 30-inch plate at 0.2 deflection is accurately known, the load supported on bearing plates of 12 to 42 inches in diameter, and for a range of deflection from 0.0 to 0.7 inch, can be calculated.

Mr. Nevitt states that the two variables, the angle of internal friction φ and cohesion c, given by the triaxial compression test, are evidence that soil strength is a two-dinensional quantity. It is to be noted from Section 14 of the paper however, that no relationship could be found between load test results and any function of <u>both</u> c and φ. On the other hand, a reasonable correlation was established between the load supported on a 30-inch plate at 0.2 inch deflection and the angle of internal friction φ, Fig. 36. That is, soil strength in these cases was what Mr. Nevitt refers to as unidimensional, rather than two-dimensional.

We would like to have considerably more load test data for bearing plates of various sizes to check the chart of Fig. 19. However, it represents the results of a large number of load tests from representative locations, and we are inclined to feel that for normal cohesive soils, if the load supported on a bearing plate of given size is known, the load supported on any other bearing plate size over the usual range of wheel load contact areas can be calculated with reasonable accuracy. On the other hand, this would not hold true for soils having abnormal load versus deflection curves. Soils of the latter type are probably uncommon in Canada, and may tend to be unusual elsewhere, but the possibility of their occurrence should be considered by both highway and airport engineers.

Mr. Nevitt has taken exception to the exponential equations (14) and (15) for required thickness of base course Mr. Nevitt's objection is theoretically sound, but we believe it is adequately answered by equations (22) and (23) in Section 27 of the paper, which could not be presented at the meeting itself due to lack of time. Our own data indicate that equations (14) and (15) may be adequate for flexible pavement design for highways, and for runways for moderate wheel loadings. Only when the higher aeroplane wheel loads are to be carried over the poorest subgrade soils would it appear to be necessary to consider equations (22) or (23).