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NO. 4 B

AIRPORT RUNWAY EVALUATION IN CANADA

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IN

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BY

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Highway Research Board
Division of Engineering and Industrial Research
National Research Council
Washington 25, D.C.

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AIRPORT RUNWAY EVALUATION IN CANADA [1]

BY

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

SYNOPSIS

This paper outlines the results of an investigation of the runways at a number of Canada's principal airports, which was conducted by the Department of Transport during 1945 and 1946. The program of testing included: a pedological soil survey and the preparation of a pedological soil map for each airport site; field moisture and density tests in place on the base course and on each 6-in. layer of the upper 18 to 24 in. of the subgrade; securing large disturbed samples of base course and of each layer of subgrade for physical and compaction tests in the laboratory, and undisturbed samples for CBR (both field and soaked condition), triaxial compression, shear, and consolidation tests; cone bearing and Housel penetrometer tests on layers of subgrade in the field; plate bearing tests (repetitive) on subgrade, base course, and surface, to determine the load supporting values of the runways, and to obtain information required for the design of either rigid or flexible pavements.

Correlation of the pedological soil map with load test data, demonstrates the connections between soil types and soil engineering properties.

The field moisture and density data indicate that saturation of the subgrade occurred at relatively few test locations.

A straight line relationship for unit load versus $\frac{P}{A}$ ratio, applies to plate diameters over a range of 12 to 42 in. and probably beyond.

Useful correlations are indicated by means of which limited load test data for a single bearing plate can be extrapolated to other bearing plate sizes between 12 and 42 in. in diameter, and to any deflection over the range of at least 0 to 0.7 in.

Base course support per unit of thickness may be generally independent of the composition of granular base course materials, but it appears to be influenced by base course density.

Bituminous surfaces seem to have greater load supporting capacity per unit of thickness than do granular bases. The ratio appears to vary from about 1.5 for those made with liquid asphalt and soft asphalt cement, etc., binders, to about 2.5 for well designed and constructed asphalt concrete, penetration macadam, and sheet asphalt.

A method for designing bituminous paving mixtures by the triaxial compression test is outlined.

The influence of repetitive loading and bearing plate size on the value of the subgrade modulus *k* is shown.

The load test data indicate that the supporting value of a given thickness of granular base at any specified deflection depends directly upon the degree of subgrade support, and this leads to a method of design for obtaining the thickness of

[1] Presented at the 26th Annual Meeting of the Highway Research Board, Dec 5 to 8, 1946.

granular base required for supporting wheel loads of any magnitude.

Charts of thickness design curves for a wide range of wheel loads have been prepared to indicate the required thickness of granular base for runways, for taxiways, aprons, and turnarounds, and for highways, based upon plate bearing tests, and upon cone bearing, Housel penetrometer, field CBR, and triaxial compression tests.

General equations of design for required thickness of flexible pavements have been developed, based upon subgrade support, base course support per unit of thickness of base, and applied wheel load.

This paper outlines the results obtained from an investigation of the runways at a number of Canada's principal airports, which has been conducted by the Department of Transport during 1945 and 1946, and describes the test procedures employed. With very few exceptions, the Department of Transport has been responsible for the construction of all airports in Canada.

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 test data 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 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 CBR, which might be correlated with plate bearing test results. Load testing is costly, the equipment is cumbersome to move from one airport site to another, and it provides a questionable basis of design for new sites where equilibrium subgrade moisture and density conditions do not exist. If it could be done with reasonable accuracy, the substitution of one or more of these simple tests for the load test, would be of considerable practical value.

5. To secure large disturbed samples of base course, sub-base, and subgrade, on which the usual physical tests, mechanical analysis, and compaction tests could be made, and undisturbed samples of the subgrade for CBR (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 load test data, to establish an equation or set of curves for required thickness, which could be employed with reasonable confidence for the design of flexible pavements to support airplane wheel loadings of any magnitude.

In view of the quantitative design recommendations for thickness of flexible pavements for airport runways, which have been advanced by the Civil Aeronautics Administration, United States Corps of Engineers, Public Roads Administration, and other organizations and authorities in the U.S.A. in recent years, often after large expenditures for comprehensive investigations, it might be considered that Item 7 of the objectives listed above, represents an unnecessary duplication of effort on our part.

Canadian highway and airport engineers greatly appreciate the tremendous amount of past and current investigational work performed on all phases of pavement design and construction by various United States organizations, and the valuable results that have been made avail-

able. However, on the basis of their own experience during the past ten years, the engineers of Canada's Department of Transport are firmly convinced that some of the thickness data for airport runways being advocated in the U.S.A. at the present time are unnecesarily conservative. Furthermore, they believe that they have sufficient traffic data of their own on which to base a reasonable opinion concerning the adequacy of any suggested runway design.

Dorval airport at Montreal is one of the hubs of air transport between North America and Europe, and was used quite extensively during the war for the ferrying of four-motored aircraft from this continent to Britain.

The overall thickness of flexible surface, base course, and sub-base at Dorval, is about 14 in. The clay sungrade has an average CBR 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 runways at Dorval would be considered unsafe for capacity operations for wheel loadings exceeding the following values for the design criteria of the respective organizations:

5,000	pounds for	USED	(*1*) [2]
7,500	" "	CAA	(*2*)
10,000	" "	PRA	(*3*)

By actual traffic count at Dorval from Jan, 1942 until Oct. 1946, the operations by aircraft of the gross loadings indicated were as follows: (each take-off or each landing is counted as one operation)

No. of Operations	Airplane Weight lb
Over 200,000	25,000 or more
" 83,000	50,000 " "
" 19,000	65,000 " "

In one day, recently, there were 77 operations by Constellations, which weigh from 80,000 to 90,000 lb.

The field CBR 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 of Dorval airport had been designed on the basis of the soaked CBR rating of the subgrade, the USED design charts indicate that an overall thickness of sub-base, base course, and pavement, of approximately 30 to 35 in. would have been required to support the wheel loadings which it has been carrying with its present thickness of 14 in.

The Department of Transport's experience at Dorval can be verified by that at many other airports in Canada. In Table 1 below, certain descriptive characteristics and traffic information are summarized for Malton Airport at Toronto, Stevenson Field at Winnipeg, and the airport at Lethbridge, Alberta, which are among Canada's busier airfields.

For Toronto and Winnipeg airports, the total number of operations of planes weighing 7500 lb or more is correct as shown. Because of the manner in which the traffic data were recorded, it has been necessary to break this information down in terms of airplanes of different weight categories, on the basis of the informed estimates of the traffic control tower operators. However, from flight schedules, and the intimate knowledge of the control tower operators, it is believed that the breakdown of traffic data given in Table 1 for Toronto and Winnipeg airports is reasonably correct. Table 1 indicates that the runways at these three airports

2 Italicized figures in parentheses refer to list of references at the end of the paper.

TABLE 1

TRAFFIC DATA FOR TORONTO, WINNIPEG AND LETHBRIDGE AIRPORTS
JAN 1, 1941 TO OCT 31, 1946

Airport	Overall Thickness Pavement and Base in.	Average CBR Value Soaked Subgrade Samples	Wheel Load Rating USED Design Curves	Actual Traffic Data to Nearest Full Thousand. Number of Operations of Aircraft Weighing More Than 7,500 lb	15,000 lb	25,000 lb	50,000 lb	64,000 lb
Toronto	8 to 10	3.5	2000 (approx)	293,000	73,000	38,000	3,400	3,000
Winnipeg	8 - 2 rwys. 14 - 1 rwy.	3.3	2000 5000	310,000	87,000	19,000	several hundred	
Lethbridge[a]	6 to 8	4.6	2000	227,000	34,000	4,000	several hundred	

[a]Traffic data for period Jan 1, 1942 to Oct 31, 1946.

have been supporting airplane wheel loadings which exceed by several times their rated safe loading according to some current U.S. designs.

At a large number of Canada's other airports, the CBR rating of soaked samples of the subgrade is, or would be, about 3 to 4, and the overall thickness of flexible surface and base is only from 6 to 10 in. While according to some U.S. designs these runways are capable of carrying wheel loads of less than 5,000 lb, a number of them have carried limited traffic by four-motored airplanes with wheel loadings of 25,000 to 30,000 lb or more.

In spite of the comparisons which have just been made, it was realized that the relatively thin base and surface on the runways at many Canadian airports probably could not withstand highly concentrated traffic by 4-motored aircraft. At the same time, Canadian engineers believe as a result of their own experience, that the thin pavements on these runways have a considerably greater load carrying capacity than their rating according to several current U.S. designs would indicate. In particular, it was felt that a design based on the CBR rating of soaked subgrade samples could not ordinarily be justified for airport runway construction in Canada. It was because of their conviction on this matter of design, that the principal engineers of the Department of Transport took the necessary steps to have the current investigation undertaken in the early spring of 1945.

It is emphasized that in starting this program of runway testing, the Department of Transport had no theories of pavement design of its own to either prove or disprove. The principal objective was to obtain the necessary test data, and let this information speak for itself. This principal has been followed consistently throughout the entire investigation.

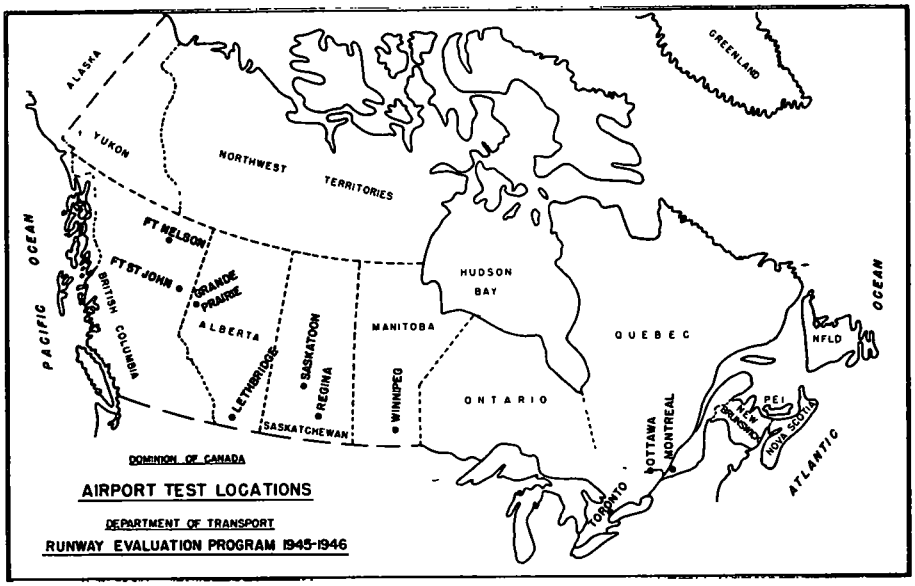

Figure 1

LOCATION AND BRIEF DESCRIPTION OF AIRPORT PROJECTS TESTED

Figure 1 indicates the locations of the ten airports which have been investigated up to the present time. It will be observed that their geographical distribution covers a very wide area extending from Eastern Canada to the southwestern approaches to Alaska.

It is emphasized that whatever progress has been made in certain principal aspects of this investigation became possible only because test data had been determined by means of identical equipment and test procedures at a considerable number of airports where climatic and soil conditions varied over a wide range. Generally speaking, it is questionable whether more than a few worthwhile results capable of being widely applied elsewhere could have been obtained from even the most concentrated study of a single airport, since the test data from a single airfield usually appear on a graph as a cluster of points, which often have little tendency to indicate unmistakable trends between the various relationships being investigated.

Grande Prairie, Fort St. John, and Fort Nelson are part of the Northwest Staging Route, and were built for the ferrying of airplanes, personnel, and supplies to Alaska and beyond during the war. The other seven airports have been a part of Canada's system of air services for some time, as regular ports of call on the schedules of Trans-Canada Airlines. The runways at these sites were constructed either before or during the early stages of the war.

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 reached approximate equilibrium insofar as the distribution of soil moisture was concerned. A general description

of the subgrade, sub-base, base course, and pavement for each of the ten airports is contained in Table 2.

At Uplands airport at Ottawa, the subgrade consists of 80 ft of clean sand, and at Fort Nelson there is from 3 to 5 ft of clean sand over clay. At the other eight airports the subgrade is clay or clay loam, with CBR values (soaked) varying from 2 to 4.5.

It will be observed that the runways at all of the airports tested so far have flexible pavements. The design for rigid pavements has recieved a great deal of study over the years and it seemed unlikely that an investigation of our own would add anything worthwhile 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 apparent inherent difficulties involved, and it was in the field of flexible pavements that the experience of airport engineers in Canada appeared to be at such variance with the design requirements advocated by principal organizations in the U.S.A.

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 and base that should be selected.

All testing was conducted in a manner that would provide 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.

SOIL SURVEY AND PEDOLOGICAL SOIL MAPS FOR AIRPORT SITES

Pedological soil surveys were made of the various airport sites by qualified soil surveyors, provided through the courtesy of the Central Experimental Farm at Ottawa, and the Soils Department at the University of Saskatchewan. From the soil surveys, pedological soil maps were prepared showing the area occupied by each soil type (Fig. 2).

Generally speaking, not more than one or two principal soil types occurred at each airport site, and most frequently there was only one. Figure 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 consisting 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 glacial till deposited during the construction over the fluvial or transition soil types. Small areas of sand and muck soils also occur.

In Table 3, a comparison is made between values of subgrade modulus for the fluvial and glacial till soils. Although both soils are within the same PRA classification range, Table 3 indicates a higher subgrade modulus for the glacial than for the fluvial soil, when both are in the undisturbed condition. The subgrade modulus of the glacial soil in embankment is considerably less than in cut, probably because of insufficient compaction.

In Table 4, a similar comparison

TABLE 2

GENERAL DESCRIPTION OF AIRPORT SITES

Airfield	Depth to Water Table	Pavement	Base Course	Sub-base	PRA Classification	Subgrade LL Av	Subgrade PI Av
Ft. St. John	Deep	3.5 to 6 in. RC 4 and 150-180 Pen. Bituminous	5.5 to 10 in. Crusher Run Gravel	5 to 17 in. Pit Run Gravel	A-7	49.2	27
Grande Prairie	Deep	2 to 8 in. SC 5 Bituminous Mixture	6.5 to 16.5 in. Mechanical Stabilization and Gravel	None	A-7	63.9	38.5
Saskatoon	Deep	1.5 to 3.5 in. SC 5 Bituminous Mixture	4.5 to 6.5 in. Gravel	None	A-7 A-6	46.5	23.7
Lethbridge	Deep	1.5 to 3.5 in. SC 5 Bituminous Mixture and Surface Treatment	4 to 7.5 in. Gravel	None	A-7 A-6	39.5	20
Dorval (Montreal)	4 to 6 ft	4 to 6 in. Pen. Macadam with Sheet Asphalt top	3 to 5 in. Water Bound Macadam	3 to 9 in. Pit Run Gravel	A-4 A-7 A-6	33.7	13.4
Winnipeg	Deep	3 to 4 in. SC 5 Bituminous Mixture	5 to 10.5 in. Mechanical Stabilization	None	A-7	64.4	36.7
Malton (Toronto)	Deep	3.5 to 9 in. SC 5 Bituminous Mixture	1.5 to 7 in. Gravel	None	A-4 A-7 A-6	32.1	13.8
Uplands (Ottawa)	Deep	2 to 3 in. SC 5 Bituminous Mixture	2.3 to 6 in. Gravel	None	A-2	18.8	0
Ft. Nelson	2 to 3 ft	4.5 to 5.5 in. 150-180 Pen. Bituminous Mixture	6 to 10 in. Pit Run Gravel	None	4 to 5 ft of Sand over Clay	24.8 (for clay)	10.4
Regina	Deep	0.5 to 1 in. Surface Treatment	5 to 7 in. Mechanical Stabilization	None	A-7	72	38

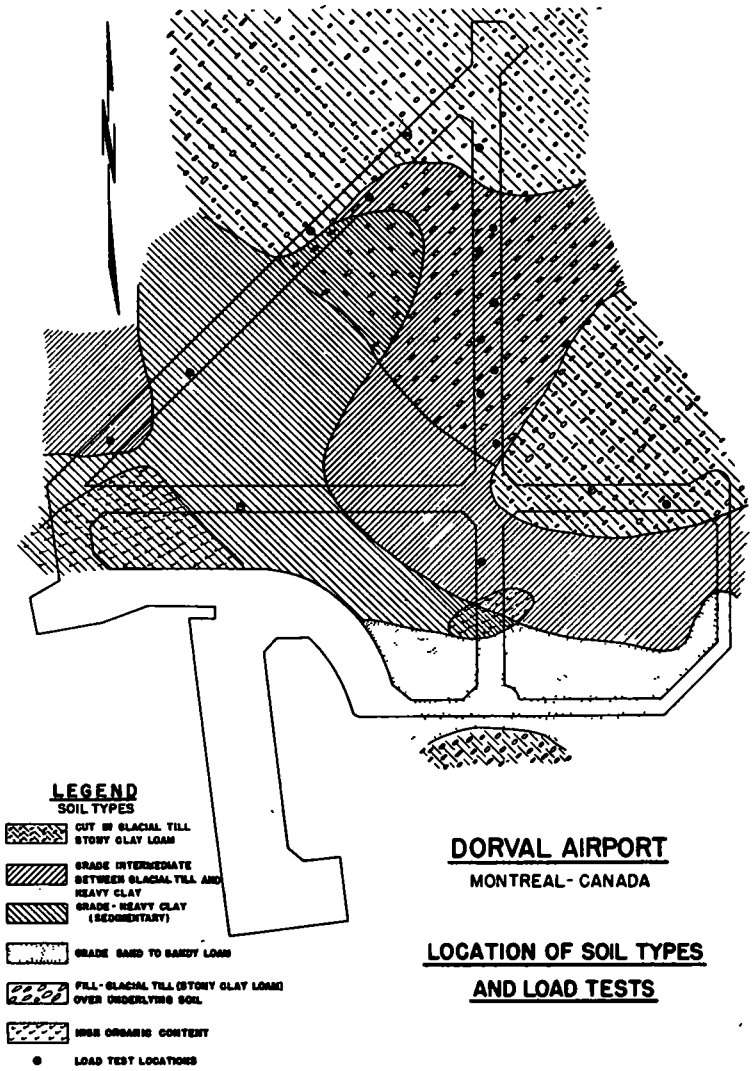

Figure 2

is made of subgrade support on a 30-in. diameter plate at 0.5-in. deflection, for the fluvial and glacial soils.

It is apparent from Table 4 that the glacial till soil at Dorval has appreciably greater bearing capacity than the fluvial material. The considerable difference in supporting value between cut and embankment sections indicated in Table 3 for the glacial till soil, has practically disappeared at a deflection of 0.5 in. While both fluvial and glacial soils fall into the same range of PRA classification, the glacial soil contains an appreciable percentage of fine gravel. The average PI of the glacial soil was about 11, and the average PI of the fluvial soil was about 20.

The information of Tables 3 and 4 emphasizes the value of the pedological soil survey to airport and highway engineers for indicating the areas occupied by soils with different engineering properties.

TABLE 3

COMPARISON OF SUBGRADE MODULUS FOR FLUVIAL AND GLACIAL TILL SOILS AT DORVAL AIRPORT

No. of Repetitions of Load	Subgrade Modulus lb per sq in. per in.		
	Fluvial Soil A-4 to A-7 Grade	Glacial Till A-4 to A-7 Cut	Glacial Till A-4 to A-7 Embankment
1	140	260	170
10	130	240	140
100	125	220	125
1000	120	210	110
10000	110	200	100

TABLE 4

COMPARISON OF SUBGRADE SUPPORT ON 30-IN. DIAMETER BEARING PLATE AT 0.5-IN. DEFLECTION FOR FLUVIAL AND GLACIAL TILL SOILS AT DORVAL AIRPORT

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

RESULTS OF FIELD MOISTURE AND DENSITY TESTS

In a number of technical articles during the past four or five years, the claim has been advanced that all subgrades and base courses may become saturated and that designs for pavement thickness should be based upon this anticipated condition. During the Department of Transport's investigation, field moisture and density tests were made in place on the base course, and on each 6-in- layer of the subgrade to a depth of 18 in., and frequently to 24 in. 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 AASHO compaction. From test data obtained in the field and on the samples sent to the laboratory, it is possible to express the field density as a percent of modified AASHO maximum density, and field moisture as a percent of modified AASHO optimum moisture. The degree of saturation in place can also be determined from the specific gravity of the soil and from field moisture and density tests. In addition, the field moisture content can be calculated as a percent of the plastic limit, and of modified AASHO optimum moisture. This information is summarized in Figs. 3 to 7 respectively.

For the eight airports with cohesive subgrade soils, relationships were determined for field density as percent of modified AASHO maximum density versus field moisture as percent of modified AASHO optimum moisture (Fig. 3); for percent saturation (field condition) versus field density as percent of modified AASHO maximum density (Fig. 4); for percent saturation versus field moisture as percent of modified AASHO optimum moisture (Fig. 5); for field moisture versus plastic limit (Fig. 6); and for field moisture versus modified AASHO optimum moisture (Fig. 7).

Figure 3 indicates that field density as expressed as a percent of modified AASHO maximum density, has little influence on field moisture expressed as a percent of modified AASHO optimum moisture. The field moisture content may be high or low, regardless of variations in field density. It also indicates that the field moisture varies over a range from about 60 to about 200 percent of modified AASHO optimum moisture.

Figure 4 shows that there may be

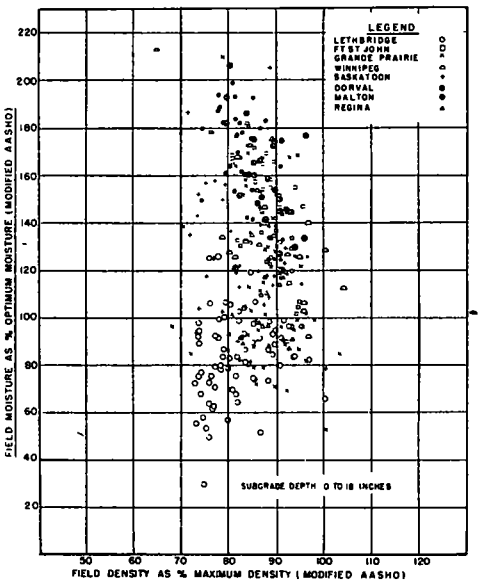

Figure 3. Field Moisture as Percent Optimum Moisture Versus Field Density as Percent Maximum Density

some tendency for the percent of saturation of the subgrade to increase, with an increase in the field density of the subgrade, when the latter is expressed as a percent of modified AASHO maximum density. The field density ranges from about 75 to about 95 percent of modified AASHO maximum density, with an overall average of about 85 percent. Apart from a certain amount of compaction at Regina and Winnepeg airports, no attempt had been made to Proctor compact the subgrade during construction at any of the sites included in the investigation.

From Fig. 5 it will be observed that the degree of saturation tends to be greater as the field moisture expressed as a percent of optimum moisture increases. It is obvious from both Figs. 4 and 5 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 percent saturation are considered to represent complete saturation, the percentage of locations that could be considered to be saturated is only 21.7 percent of the total.

If the field moisture content of the subgrade was always exactly equal to the plastic limit, all points in Fig. 6 would have fallen along the 45 degree line labelled "100 percent PL". This graph indicates that in 63.8 percent of the locations tested, the field moisture is less than the corresponding plastic limit. Broken lines have been drawn through the data of Fig. 6 to represent 60, 70, 80, 90, 100, 120, and 130 percent of the plastic limit. In a new location where no pavement exists and the anticipated subgrade moisture cannot therefore be measured, this information is useful for estimating the probable subgrade moisture content to be expected, provided always that the methods of construction are similar to those employed for airport subgrades in the past. It is quite conceivable that the field moisture contents of Fig. 6 would be considerably reduced if the subgrades were compacted to high density.

For the methods of subgrade construction employed for these eight

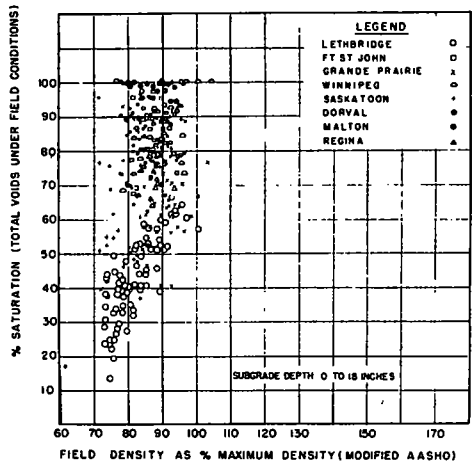

Figure 4. Percent Saturation Versus Field Density as Percent of Maximum Density

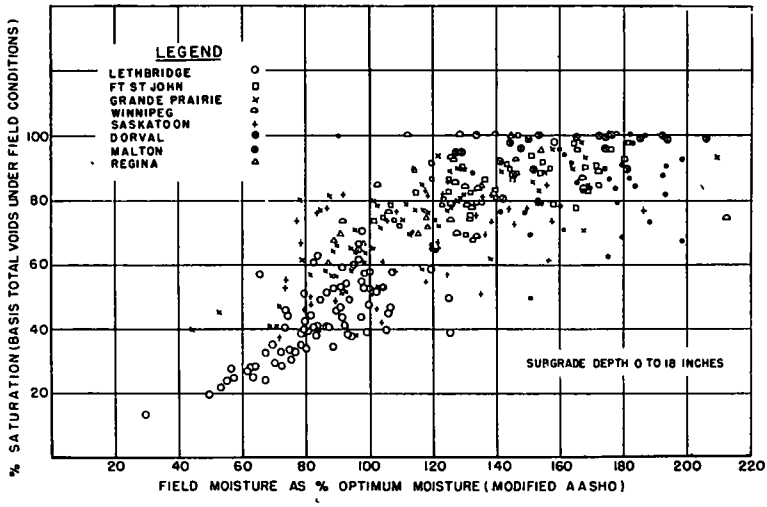

Figure 5. Percent Saturation Versus Field Moisture as Percent Optimum Moisture

airports, which included little or no compaction, and for field conditions similar to those at airports on clay subgrades in wetter climates in Canada, the equilibrium moisture content to be designed for could be as high as 120 percent of the plastic limit. For climatic and drainage conditions represented by Lethbridge airport, on the other hand, a moisture content equal to about 80 percent of the plastic limit is the maximum to be anticipated for the subgrade for runways.

Kersten(*4*) 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.

Figure 7 indicates that the field moisture content of the clay subgrades at the eight airports exceeded the modified AASHO optimum in 71.2 percent of the locations tested. This graph can also be usefully employed when estimating the probable subgrade moisture content to be expected under paved runways in a new location.

One of the principal problems encountered when designing pavement thicknesses for runways in a new location concerns the moisture which a clay subgrade soil will eventually contain. The plastic limit and optimum moisture can be easily determined in a laboratory on representative samples of soil from the new site. With this laboratory information, and a knowledge of the drainage and climatic conditions to which the subgrade at the proposed location will be exposed, a reasonable estimate of the final equilibrium subgrade moisture content can be made by reference to Figs. 6 and 7. When the subgrade is to be highly compacted to a depth of 2 to 3 ft, the ultimate field moisture would probably be less than that indicated in Figs. 6 and 7.

PLATE BEARING TESTS

Equipment: To determine the support-

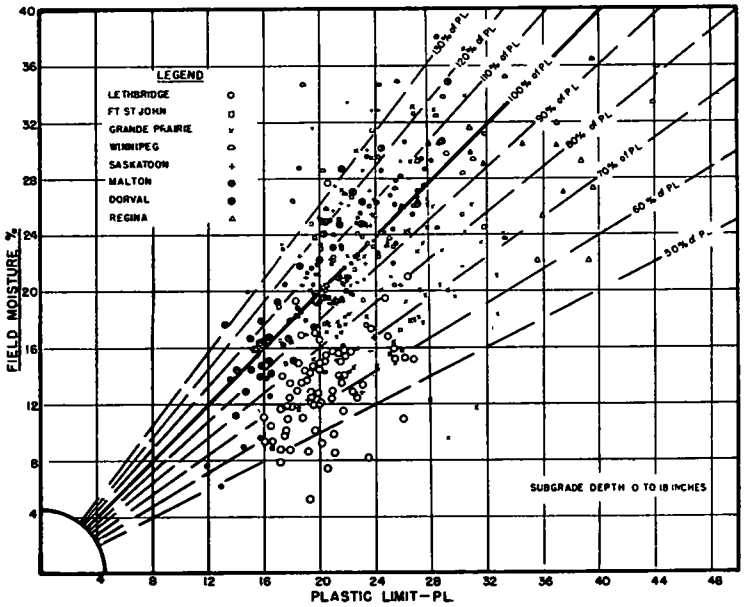

Figure 6. Field Moisture Versus Plastic Limit - P.L.

ing capacity 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 Highway Research Board(5).

In 1945, four weighted tractor trailer units capable of applying loads of from 70,000 to over 100,000 lb were employed as the source of reaction. Two of these units are shown in Figs. 8 and 9. During 1946 only one of these units (Fig. 9) was in operation.

The arrangement of equipment for performing each load test is illustrated in Fig. 10.

Circular steel plates 1-in. thick and 30 in. in diameter were used for most tests, but a considerable number were performed with bearing plates 12, 18, 24, 36, and 42 in. 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-lb capacity. The jacks were equipped with gauges graduated in increments of 1,000 or 2,000 lb. A spherical bearing was placed between the top of the jack and the jacking point on the trailer.

Deflections of the bearing plate were measured to the nearest 0.0001 in. by means of two Ames dials graduated in increments of 0.001 in. set on the plate near the extremeties of a diameter. For the 1946 portion of the investigation, deflections of the adjacent surface of the pavement were determined by three additional Ames dials, spaced at 0.5, 1.0, and 1.5 diameters beyond the perimeter of the bearing plate in use (Fig. 10). These three latter deflection gauges rested on the heads of short nails driven flush with the pavement surface.

The Ames dials were supported from adjustable steel arms attached to a deflection beam consisting of an 18-ft length of 2- or 2½-in. diameter standard pipe, resting at its extremities on broad based stands. During the test, the long axis of the deflection beam was at right angles to the longitudinal axis of the loaded trailer. All points of support for either the tractor-trailer units, or for the

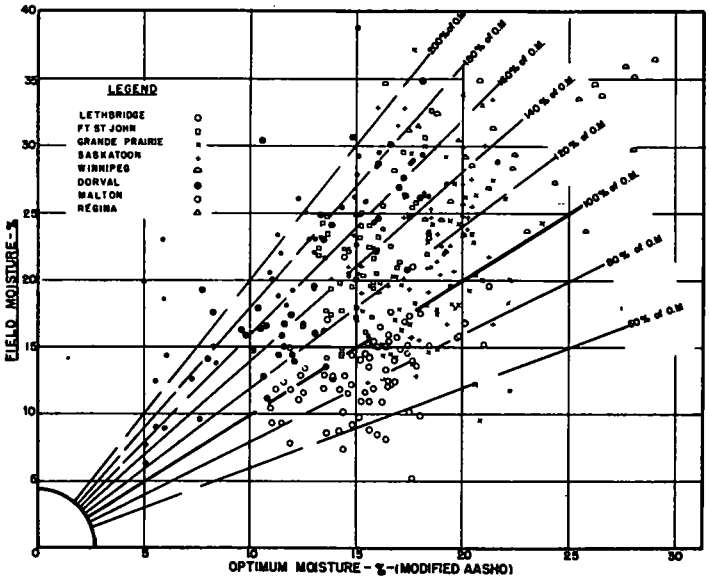

Figure 7. Field Moisture as Percent Versus Optimum Moisture as Percent (Modified AASHO)

deflection beam were at least 8 ft from the bearing plate.

Load Test Procedure: The load test procedure employed, while following in general that recommended by the Highway Research Board Committee on Flexible Pavement Design(5) was also governed by the need for obtaining the test data required for the design of either rigid or flexible pavements.

It was therefore necessary to employ one loading of a magnitude which would give a deflection of approximately 0.05 in., from which the subgrade modulus for rigid pavement design could be determined. Another load, giving a deflection of about 0.5 in. 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.

It was realized that 0.05 in. for rigid pavement design, and 0.5 in. for flexible pavement design may not be the critical deflections that should be employed in all cases. It has been suggested that the critical deflection for the subgrade under flexible pavements depends upon the thickness of overlying sub-base, base course, and surfacing(6) and that the critical deflection for a flexible pavement itself depends upon its radius of curvature under load. However, until more is known concerning the role of these and other factors, the above critical values appear to be the most reasonable that can be adopted at the present time, and they are in quite common use.

After some experimenting, a test procedure was standardized which is here outlined briefly:

After the equipment had been set up quick loads of 1000, 2000, 3000, 4000, 5000, and 6000 lb were applied once to the 12-, 18-, 24-, 30-, 36-, and 42-in. plates respectively, and immediately released. This was done to obtain better seating of the plates on each other and on the loaded area.

The deflection gauges were zeroed at loads of 500, 1000, 1500, 2000, 2500 and 3000 lb for bearing plates of 12, 18, 24, 30, 36, and 42 in. in diameter,

respectively.

A load giving a deflection of about 0.03 in. was applied, a stop watch started and the same load maintained until the increment in deflection was 0.001 in. or less per min for each of three successive minutes.

The load was then completely released and the deflections were recorded until the rate of recovery was 0.001 in. or less per minute for each of three successive minutes.

The same load was applied and released in this manner six times.

Figure 8. Load Test Unit No. 1. Capacity 150,000 lb

The load was increased to give a deflection of about 0.2 in., and the application and recovery of this load repeated from 4 to 6 times. In all cases the standard end point was taken to be 0.001 in. or less per min for three successive minutes.

The load was finally increased to provide a deflection of from 0.35 to 0.40 in., and repeated as before from 4 to 6 times.

Deflection readings for the two deflection gauges resting on a bearing plate near the extremities of a diameter, were read at the end of every minute and recorded in field notebooks

Readings of the Ames dials set at 0.5, 1.0, and 1.5 diameters beyond the perimeter of the bearing plate were recorded just before the release and just before the application of load for each repetition.

From a thermometer near the bearing plates the air temperature was read and recorded periodically.

An electric bell attached to the deflection beam was buzzed briefly 10 sec before the deflection gauges were read. The slight vibration generated by the bell assured good contact between gauges and bearing plate or pavement, when the readings were made.

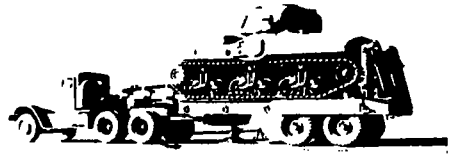

Figure 9. Load Test Unit No. 4. Capacity 100,000 lb

Similar equipment and identical load test procedures were employed by each load testing crew, in order that a common basis would exist for correlating the data obtained at each airport tested.

Load tests were made on the surface of the pavement, on the surface of the base course, and on the surface of the subgrade. Figure 11 illustrates the general arrangement followed in 1945 for the grouping of the load tests at each test location on a runway. For load tests on the base course, the pavement was removed from a circular area 12 ft in diameter, and the bearing plate was placed in the center. For load tests on the subgrade, both pavement and base course were excavated to the top of the subgrade over a circular area 12 ft in diameter, in order that the subgrade load 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. 11).

In 1946, the spacing between the individual load tests at each test

location was at least 1.5 plate diameters. For unconfined base course and subgrade tests, the pavement, and the pavement and base course, respectively, were excavated over a circular area of sufficient diameter to avoid confinement within 1.5 diameters of the bearing plate. It is believed that this provides enough clearance for routine unconfined tests. All field tests were made, and disturbed and undisturbed samples obtained within a rectangular area immediately adjacent to the bearing plate.

A load testing crew consisted of three men, one keeping notes, one reading deflection gauges, and one checking the applied load during application and release. Two shifts were operated with each load test unit, and from 18 to 20 hr per day were required to complete three load tests (three magnitudes of load for each test). One field testing and sampling crew was provided with each load test unit, to make field moisture and density determinations, to perform cone bearing and Housel penetrometer tests, and to obtain the disturbed and undisturbed subgrade samples to be sent to a central laboratory.

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 100 employees for the various phases of work involved in the field and laboratory testing.

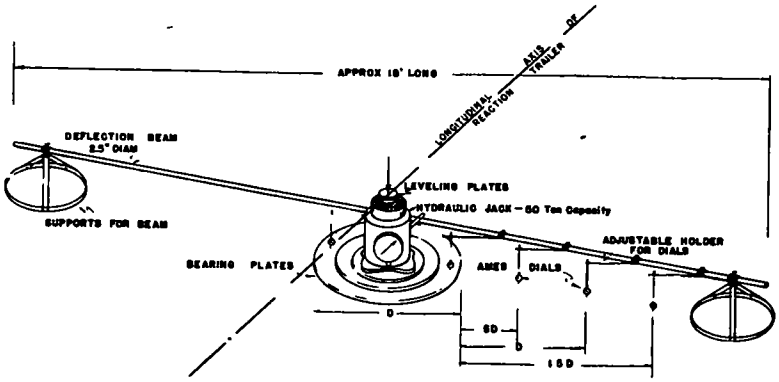

Figure 10. Diagram Showing Arrangement of Equipment for Bearing Test

Plotting of Load Test Data for Load Deflection Curves To obtain the data needed for the construction of load versus deflection curves, the following steps were involved:

For each repetition of each load, the deflection was determined at which the rate of deflection was exactly 0.001 in. per min. This can be found with sufficient accuracy from inspection of the deflection data for each repetition of load recorded in the field notebooks.

By means of a calibration curve for jack gauge readings versus the load registered by a standard testing machine during a calibration test for each jack and pressure gauge used, a correction is made to the recorded loads as read from the pressure gauge of each hydraulic jack employed.

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 1000 to 3000 lb. The zero point correction for deflection is obtained graphically (Fig. 12), and occasionally may amount to 0.02 or 0.03 in. It must be added algebraically to the observed deflections.

Zero point corrections are particularly important for the determination of subgrade modulus for rigid pavement design, since considerable error may occur if they are not made.

The corrected deflections (at which the rate of deflection is exactly 0.001 in. per min 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. 13).

There may be some question concerning the validity of extrapolating the lines in Fig. 13 to 100, 1000, and 10,000 repetitions of load, from data for 4 to 6 repetitions. This relationship was checked at a considerable number of locations for 100 repetitions of load, with the 30-in. diameter plate, on surface, base course, and subgrade. The results of one of these, which can be considered generally representative, are shown in Fig. 14, and indicate that the relationship holds reasonably well up to 100 repetitions of load. Up to this number of repetitions, the difference is very small whether the data are plotted on semi-log or log-log paper, but in general the straight line relationship seems to hold best for the log-log graph. It should also be added that in some tests the direction of the curve has become somewhat uncertain for the last 10 or 15 repetitions. Most of the information presented in this paper is based upon 10 repetitions of load, and for this small number of repetitions it makes no practical difference whether a semi-log, or log-log graph of deflection versus number of repetitions of any given load is employed.

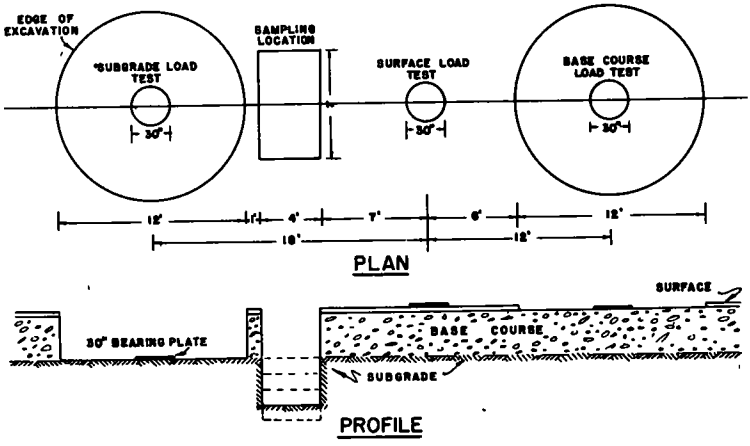

Figure 11. Diagram Showing Typical Arrangement of Load Tests and Sample Locations

Figure 15 was prepared directly from Fig. 13. The curves from top to bottom represent load versus deflection for 1, 10, 100, 1000, and 10,000 repetitions of load, respectively. From Fig. 15, 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-in. deflection, while for flexible pavement design, the load corresponding to 0.5-in. deflection can be used.

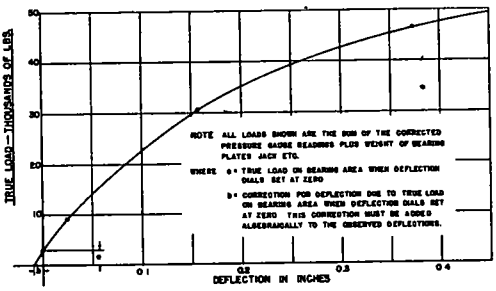

Figure 12. Procedure for Establishing Zero Point Correction for Load Versus Deflection Curves

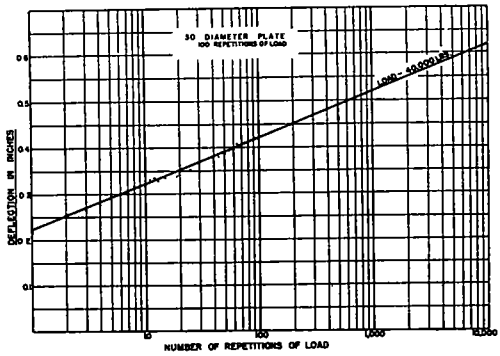

Figure 14. Increase in Deflection Resulting from Repetitions of a Given Load

LOAD TEST DATA VERSUS SAFE DESIGN FOR RUNWAY WHEEL LOADINGS

From the traffic data for several of the airports included in this investigation, it has been possible to estimate the maximum wheel loadings which the runways have been supporting under reasonably intensive traffic. This estimate is somewhat complicated by the fact that at most airports in Canada, the runways also serve as taxiways to a considerable extent.

It has been known for some time that a greater thickness of base and surface is required for taxiways, aprons, and turnarounds, than for runways, for the same airplane wheel loading. This matter is discussed quantitatively under THICKNESS DESIGN CURVES FOR TAXIWAYS, AND TURNAROUNDS later in this paper A given thickness of base and surface will therefore support a smaller wheel load as a taxiway, than as a runway. It must also be remembered that runways are wider than taxiways, and that the traffic of taxi-

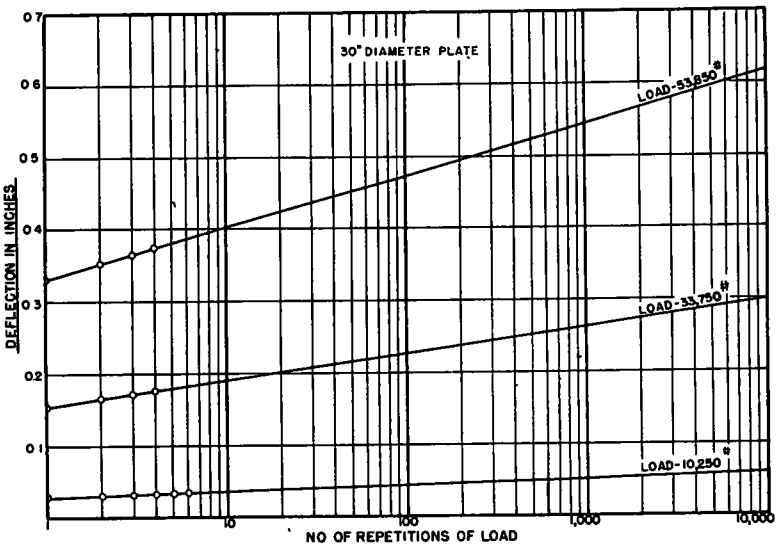

Figure 13. Influence on Deflection of Repetitions of Loading for Each of Three Magnitudes of Load

ing planes on runways may be more widely distributed and therefore less severe than on taxiways.

There is an added difficulty in that the load test results obtained for any runway are not identical, but may vary by many thousands of pounds between the low and high values. A single low value might be far out of line with the other results, and for this reason it seems undesirable to consider the lowest load test value as a basis for the load carrying capacity rating of a runway. If an average value of the load test results were selected as being representative, it might lead to underdesign for a considerable part of each runway. For these reasons, the load test result at the lower 25 percent point (the lower quartile point) was adopted as the representative load supporting value for each runway. That is, the load test value which was selected as representative for each runway was greater than 25 percent, but smaller than 75 percent of the load test results obtained

When all these factors were considered, it was found that the lower quartile plate bearing value (the lower 25 percent point) at 0.5-in. deflection for 10 repetitions of load, provided a load test value which appeared to be approximately equal to the maximum wheel load which the runways had been supporting under reasonably intensive traffic. It should be emphasized that both wheel load and representative plate bearing value must apply to the same contact area. It is worthwhile noting in this regard, that the USED have observed in their accelerated traffic investigations of test locations, that "Where failures occur in flexible pavements they occur in relatively few operations rather than over an extended number(*6*)".

It is of interest that the load supported in a plate bearing test at 10 repetitions of load for 0.5-in. deflection has been suggested by the Highway Research Board Committee on Flexible Pavement Design as a criterion of safe design for wheel loads on runways(*5*). Further information may indicate that this approach should be modified, but since it seems to fit in with present traffic experience in Canada,

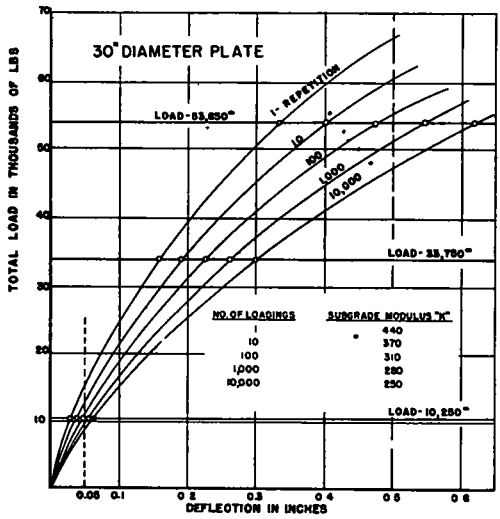

Figure 15. Load Versus Deflection for Repeated Loadings

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.

GENERAL INFORMATION FROM LOAD TEST DATA

Influence of $\frac{P}{A}$ Ratio on Unit Load Bearing Capacity: 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 Bertram(*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 ratio $\frac{P}{A}$ 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 in. Professor Housel's investigations on the other hand, have indicated that the straight line graph of unit load versus $\frac{P}{A}$ ratio holds for bearing plates up to at least 40 in. in diameter, and probably well beyond.

A considerable error in underdesign would result for the heavier airplane wheel loadings, if the unit load supporting value for a contact area of 1500 sq in. (diameter 44 in.) for example, were assumed to be the same as that obtained from a load test on a 30-in. diameter plate having a contact area of 707 sq in., but was actually considerably less.

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 in. in diameter. When the values of unit load are plotted versus the $\frac{P}{A}$ ratio for these different plates at any given deflection, graphs similar to that shown in Fig. 16 are obtained.

Due to differences in soil properties occurring under the individual plates at each group of load tests because of the spacing required between the different plates, (at least 1.5 plate diameters), or to deficiencies in test procedure, or to other variables difficult to control, it is seldom that all the points lie on the best average straight line for every deflection, although they seem to tend to do so. Figure 16, however, can be considered representative.

When all of the load tests with bearing plates of different sizes are considered, there seems to be little doubt that a straight line

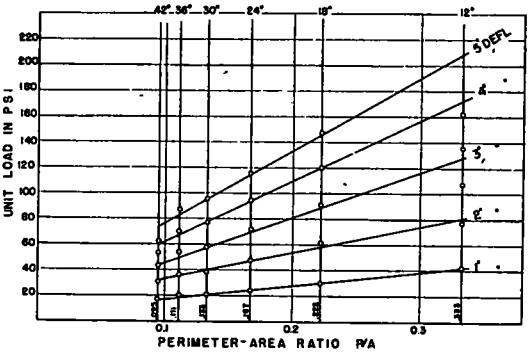

Figure 16. Influence of Plate Size on Unit Load at Different Deflections

relationship exists between unit load support at a given deflection versus $\frac{P}{A}$ ratio, for bearing plates with diameters between 12 and 42 in., and probably larger.

These results, therefore, confirm those of Housel on the influence of bearing plate size on unit subgrade support. It is also to be noted that the investigations of Campen and Smith indicate this straight line relationship for bearing plates between 23.4 and 9.6 in. in diameter(*9*), and for bearing plates between 32 and 16.6 in. in diameter(*10*).

Ratios of Loads Supported on Given Bearing Plate at Different Numbers of Repetitions: Figure 17 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-in. diameter bearing plate at 0.5-in. deflection. Similar ratios seem to hold for 10 versus 100 repetitions, and 10 versus 1000 repetitions, over the range from 0- to 0.7-in. deflection. These ratios are summarized in Table 5, for deflections between 0.2 and 0.7 in.

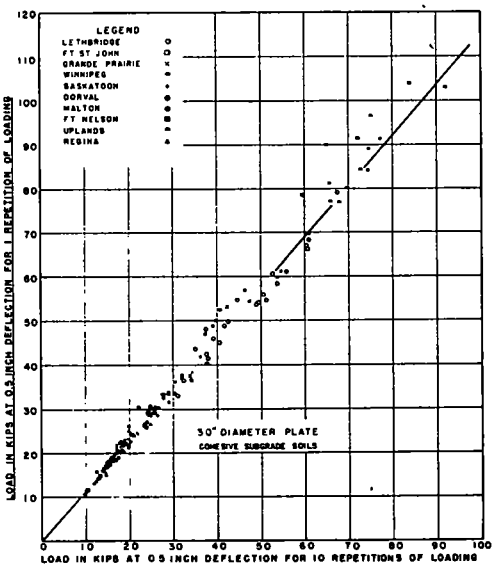

Figure 17. Load in Kips at 0.5-in. Deflection for 1 Repetition Versus Load in Kips at 0.5-in. Deflection for 10 Repetitions

The ratios of Table 5 are convenient when designing for more limited or for heavier traffic, than the load indicated for 0.5-in. deflection at 10 repetitions of load, which is employed in this paper as a criterion for safe runway design.

A study of the load test data indicates that the ratios of Table 5 are as equally applicable to bituminous surfaces as to cohesive subgrades.

TABLE 5

RATIO OF LOAD FOR "N" REPETITIONS AT DEFLECTION INDICATED TO LOAD FOR 10 REPETITIONS AT SAME DEFLECTION

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

Ratio of Loads Supported on a Given Bearing Plate at Different Deflections: When the curves for load versus deflection for load tests with the 30-in. diameter plate on the subgrades at all ten airports were analyzed, the relationships illustrated in Figs. 18, 19, 20, 21, 22, 23, and 24 were developed. This information is summarized in Fig. 25, as an arithmetic graph of the ratio of load supported at any deflection up to 0.7 in. over load carried at 0.2-in. deflection, versus deflection in inches.

Figures 25 and 26 indicate that if the exact load supported at 0.2-in. deflection can be accurately determined for a 30-in. plate, the complete load deflection curve can be calculated over the range of deflection between 0 and 0.7 in. on the basis of the information summarized in Table 6.

Relationships similar to those shown in Figs. 25 and 26 can be very easily established on the basis of deflections other than 0.2 in. For most cohesive subgrade soils, however, a deflection of 0.2 in. is

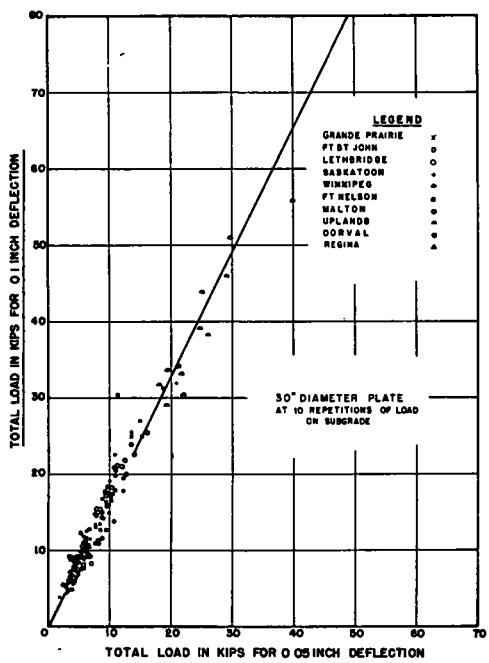

Figure 18. Total Load in Kips for 0.1-in. Deflection Versus Total Load in Kips for 0.05-in. Deflection

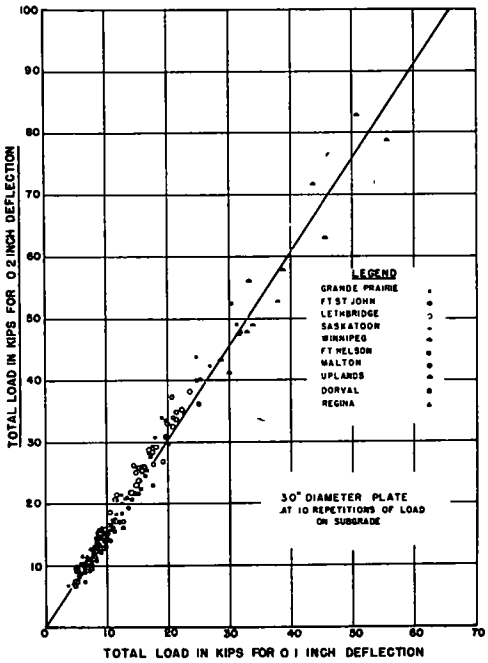

Figure 19. Total Load in Kips for 0.2-in. Deflection Versus Total Load in Kips for 0.1-in. Deflection

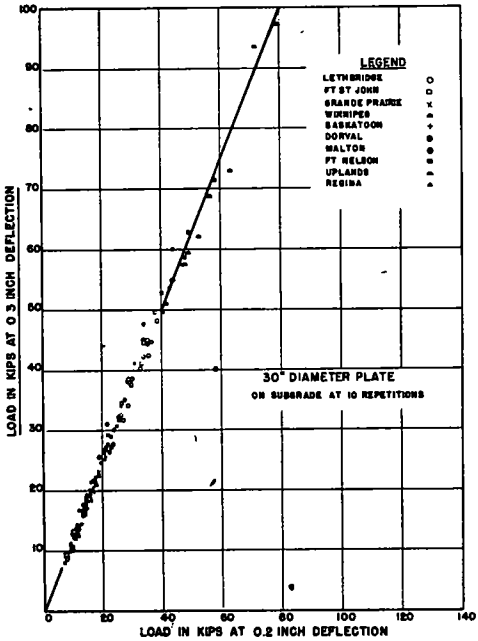

Figure 20. Total Load in Kips at 0.3-in. Deflection Versus Total Load in Kips at 0.2-in. Deflection (Subgrade)

one which can be obtained with a 30-in. diameter bearing plate, without the necessity for exceptionally heavy trailers or other sources of load. It is also usually well beyond the uncertain region of initial loads and initial settlements and belongs to what might be considered the normal portion of a load deflection curve.

It is to be noted that the relationships illustrated in Figs. 25 and 26, hold not only for subgrade load tests for the eight airports with cohesive subgrade soils, but for Uplands (Ottawa), where the subgrade consists of about 80 ft of clean sand, and for Fort Nelson, where the subgrade structure is 3 to 5 ft of clean sand over an indefinite depth of clay. It has been observed in other respects,

TABLE 6

RATIO OF LOAD SUPPORTED ON A 30-IN. DIAMETER PLATE AT ANY DEFLECTION FROM 0 TO 0.7 IN. VERSUS LOAD SUPPORTED AT 0.2-IN. DEFLECTION, FROM LOAD DEFLECTION CURVES FOR SUBGRADE SOILS

Load	carried	at	0.7-in.	deflection	is	1.906	of	that	supported	at	0.2-in.
"	"	"	0.6-"	"	"	1.768	"	"	"	"	0.2-"
"	"	"	0.5-"	"	"	1.616	"	"	"	"	0.2-"
"	"	"	0.4-"	"	"	1.443	"	"	"	"	0.2-"
"	"	"	0.3-"	"	"	1.244	"	"	"	"	0.2-"
"	"	"	0.2-"	"	"	1.000	"	"	"	"	0.2-"
"	"	"	0.1-"	"	"	0.656	"	"	"	"	0.2-"
"	"	"	0.05-"	"	"	0.401	"	"	"	"	0.2-"

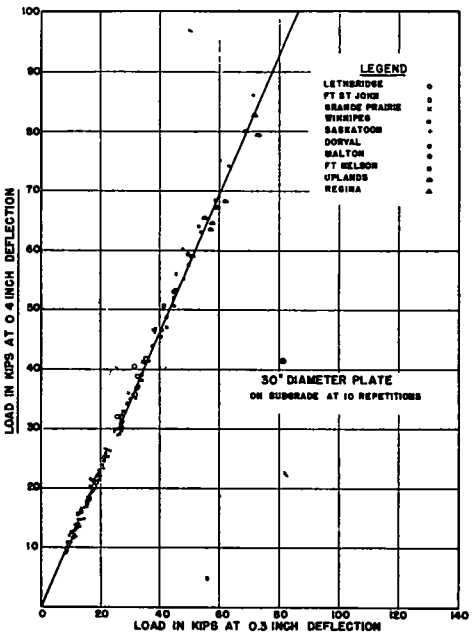

Figure 21. Total Load in Kips at 0.4-in. Deflection Versus Total Load in Kips at 0.3-in. Deflection

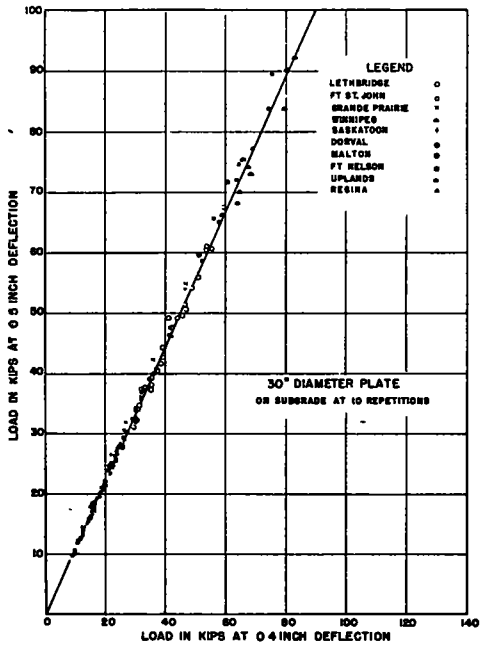

Figure 22. Total Load in Kips at 0.5-in. Deflection Versus Total Load in Kips at 0.4-in. Deflection

however, that load tests on these two airports with sand subgrades had characteristics normally associated with bearing plate tests on cohesive soils. The top layers of sand, therefore, seem to have contained a small amount of soil fines or organic matter, or both, which served effectively as a binder material.

It is to be emphasized that the ratios represented graphically in Figs. 18 to 26 represent average relationships and could only be determined as the result of a large number of load tests. From this investigation there are load deflection curves for over 200 subgrade load tests, and for over 750 load tests when those made on surface and base course are included. While this number is not particularly large, it appears sufficient to provide reasonably average results for most of the plate sizes included. Anyone who attempts to match a single load deflection curve, or any small number of load deflection curves for a 30-in. diameter plate, with the relationships shown in Figs. 25 and 26 may be disappointed by what appears to be poor agreement. If a large number of load deflection curves obtained by means of the load test procedure previously outlined, are tried, however, it will probably be found that their average curve corresponds more closely to that represented by these graphs.

The very close correlation of the data illustrated in Figs. 18 to 24 is somewhat surprising in view of the fact that these relationships were not observed until after the load test data had been obtained, and that the load tests were performed by test crews with no previous experience, working in two shifts, and operating many hundreds of miles apart.

Information similar to that in Figs. 18 to 24 has been assembled for 12-, 24-, and 36-in. diameter

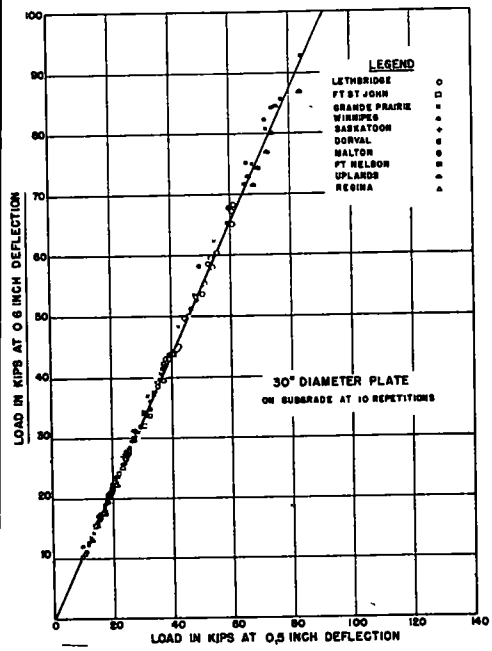

Figure 23. Total Load in Kips at 0.6-in. Deflection Versus Total Load in Kips at 0.5-in. Deflection (Subgrade)

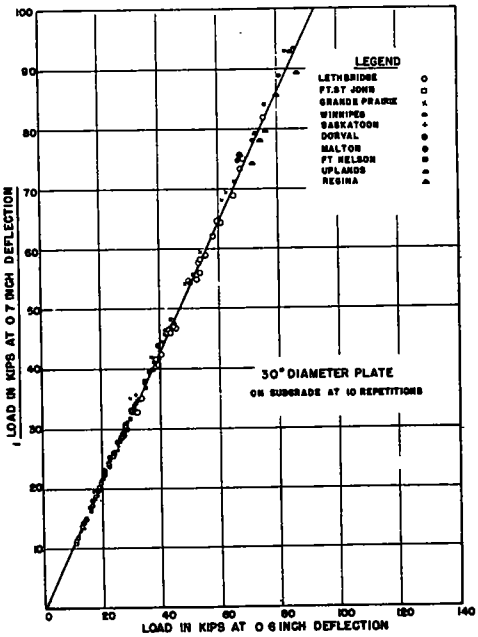

Figure 24. Total Load in Kips at 0.7-in. Deflection Versus Total Load in Kips at 0.6-in. Deflection

plates with very similar results. The present load test data for the 42- and 18-in. diameter plates are considered to be too limited to be reasonably accurate, and have not been included. An overall comparison of the relationships for the 12-, 24-, 30-, and 36-in. diameter plates is shown in Fig. 27. It may be seen that the ratios for bearing plate diameters of 12, 24, and 36 in do not coincide with those for the 30-in. plate. The greatest number of subgrade load tests were made with the 30-in. plate. If as many tests had been made with the 12-, 24-, and 36-in. plate as with the 30-in. plate, the curves of Fig. 27 might have coincided. On the other hand, Fig. 27 may be quite representative and more data might merely confirm the indications of this diagram that each bearing plate has its own set of ratios.

A certain amount of uncertainty exists concerning the exact shape of the load deflection curves in the vicinity 0.05- and 0.1-in. deflection, possibly because of initial conditions of adjustment under load. Consequently, the relationships (ratios) of Figs. 25, 26, and 27, involving portions of the curves over this range of deflection may not be entirely accurate, but it may be quite difficult to obtain more representative values.

Professor Housel of the University of Michigan has made a large number of load tests during the past 18 years. His load testing procedure consists of static loading in which each increment of load is applied for exactly 1 hr, whereas the Department of Transport employed repetitive loadings in which each application or release of load was maintained until the rate of deflection or recovery, respectively, was 0.001 in. per min or less for each of three successive minutes.

Professor Housel's data for load deflection curves for about 45 field load tests on cohesive soils, in

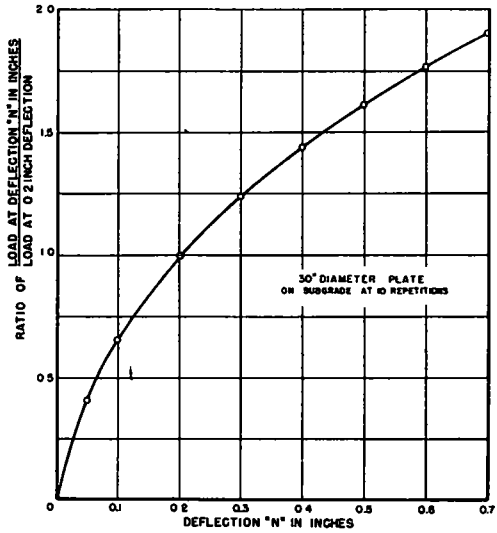

Figure 25. Ratio of Load at Deflection "N" in Inches Over Load at 0.2-in. Deflection Versus Deflection "N" in Inches

which circular bearing plates of 1, 2, and 4 sq ft had been employed (*14*) were analyzed to provide information similar to that of Figs. 18 to

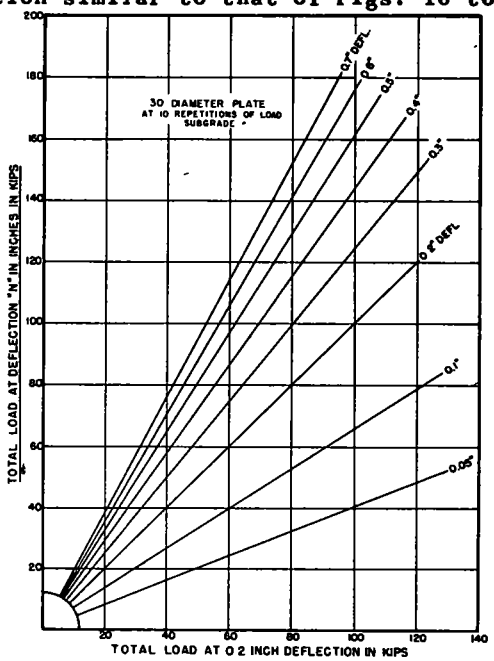

Figure 26. Load at Deflection "N" in Inches Versus Load at 0.2-in. Deflection

24. In Fig. 28, total load supported at 0.7-in. deflection has been plotted versus total load supported at 0.6-in. deflection for a circular bearing plate having an area of 4 sq ft, (Housel data). A very good straight line relationship appears to hold. Figure 24 is representative of the graphs obtained for other combinations of deflections. Similar information was determined for other deflections for the three bearing plates of 1, 2 and 4 sq ft, and is summarized in Fig. 29, wherein an overall comparison of the relationships is shown. Fig. 29 for Housel data corresponds to Fig. 27 for Department of Transport data.

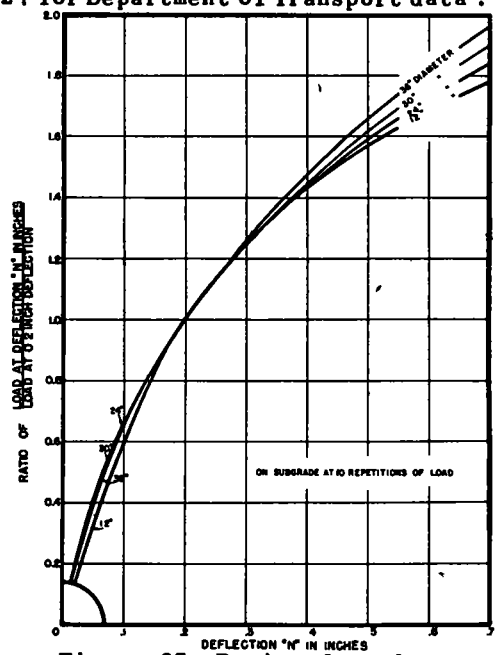

Figure 27. Ratio of Load at Deflection "N" in Inches Over Load at 0.2-in. Deflection Versus Deflection "N" in Inches for Bearing Plates 36, 30, 24 and 12 Inches in Diameter (Subgrade)

Although they are not of exactly the same area, the relationships from Fig. 29 for Housel data for a bearing plate of 4 sq ft are compared with the relationships from Fig. 27 for Department of Transport data for the 24-in. diameter plate,

and are shown in Fig. 30. It is apparent from Fig. 30 that the ratios are quite different and their unlike values probably reflect the differences in load test procedure employed in each case. Figure 30, therefore, emphasizes the fact that the values of the ratios indicated by Figs. 25, 26, and 27 will apply only when load test procedures identical (or approximately so) with those used by the Department of Transport are employed.

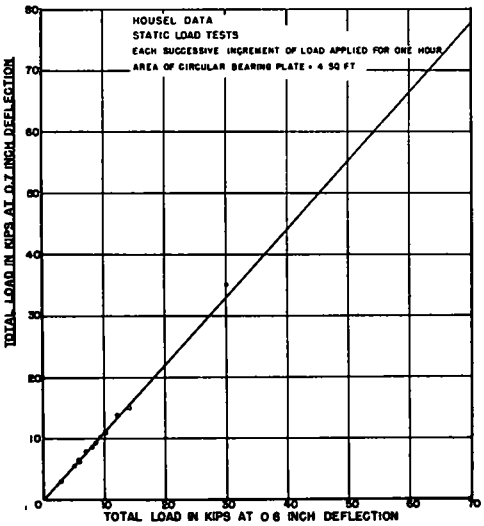

Figure 28. Total Load in Kips at 0.7-in. Deflection Versus Total Load in Kips at 0.6-in. Deflection (Housel Data)

It would appear that the ratios indicated by Figs. 25, 26, and 27 are universal constants for each bearing plate size indicated, and for the load testing procedure employed, at least for cohesive subgrade soils similar to those encountered so far in the investigation. They may also apply to granular subgrade soils. It would also seem that universal constants similar but with different values, could be developed for other load test

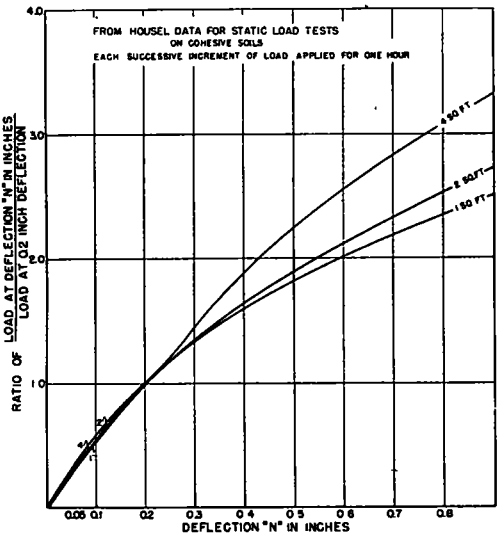

Figure 29. Ratio of Load at Deflection "N" in Inches Over Load at 0.2-in. Deflection Versus Deflection "N" in Inches for Circular Bearing Plates of 1, 2 and 4 sq ft (Housel Data)

methods, since these constants (ratios) appear to depend upon the size of bearing plate and the nature of load test procedure employed.

Similar information has been developed for load tests made on the surfaces of flexible pavements. In Figs. 31 and 32, the loads supported at 0.6- versus 0.5-in. deflection, and at 0.3- versus 0.2-in. deflection respectively, are shown for surface load tests made with a 30-in. diameter plate. This information for bearing plate diameters of 12, 24, 30, and 36 in., and for a deflection range of 0 to 0.7 in. is summarized in Fig. 33.

In Figs. 31 and 32, the points are somewhat less tightly clustered about the best average line than is the case for the corresponding graphs for the subgrade, but the trend is unmistakable. In Fig. 33, the curves for the different plate sizes do not follow the regular order, which occurred in Fig. 27 for subgrades. While the ratio of load supported at one deflection to that supported at another, for any

given plate size, is nearly the same for both surface and subgrade load tests, they are not identical. The small dissimilarity in ratios may be due to experimental error, or may be caused by fundamental differences in the behaviour of subgrade soils and bituminous surfacing materials under the stresses imposed by bearing plates during load tests.

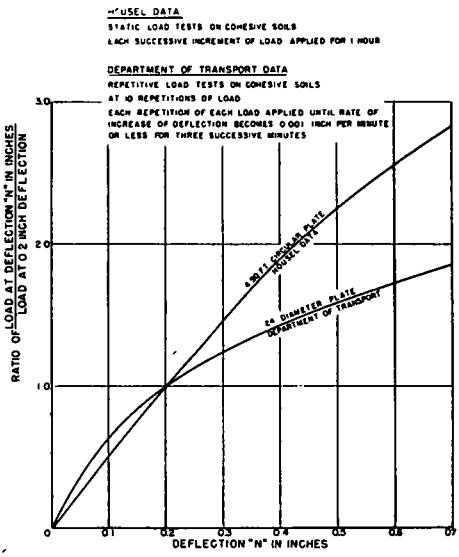

Figure 30. Comparison of Ratios of Load at Deflection "N" Over Load at 0.2-in. Deflection Versus Deflection "N" for Housel and Department of Transport Load Test Procedures

Ratios of Loads Supported on Bearing Plates of Different Sizes at Same Deflection: In Figs. 34 and 35 the total load carried on a 36-in. plate is plotted versus the total load supported by a 30-in. plate at deflections of 0.2 and 0.5 in., respectively. A straight line relationship is indicated in each case. When examining Figs. 34 and 35 it must be kept in mind that each point represents data for two load tests spaced about 12 ft apart and that each of the two tests was made

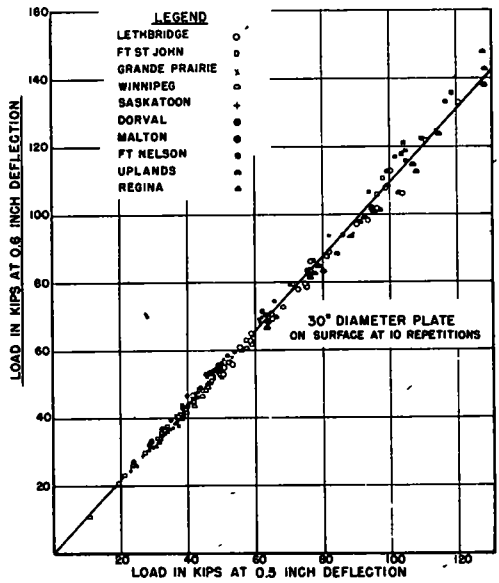

Figure 31. Load in Kips at 0.6-in. Deflection Versus Load in Kips at 0.5-in. Deflection (Surface)

with a bearing plate of different size. Anyone familiar with load testing will realize that two load tests made with even the same plate, but spaced 12 ft apart, may indicate results that differ by several

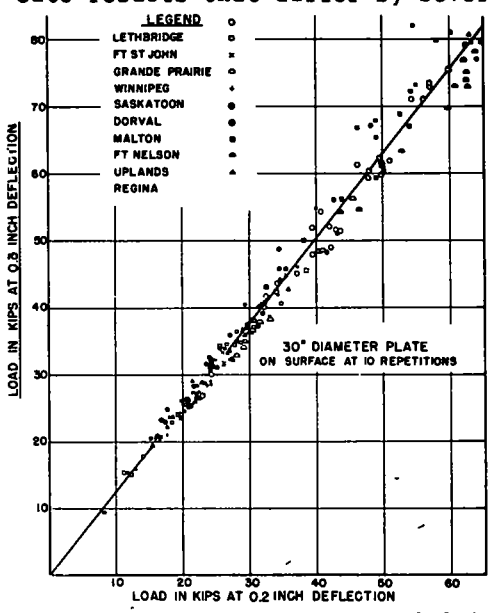

Figure 32. Load in Kips at 0.3-in. Deflection Versus Load in Kips at 0.2-in. Deflection (Surface)

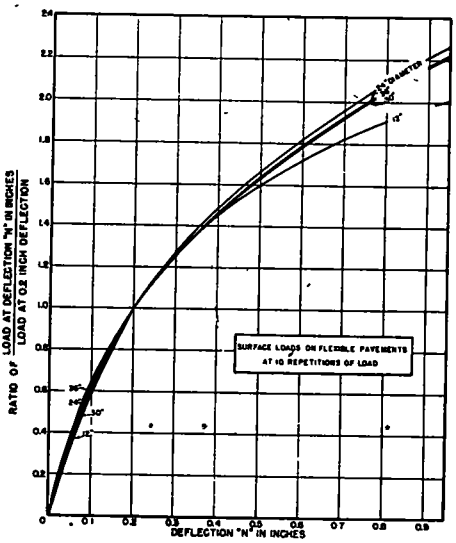

Figure 33. Ratio of Load at Deflection "N" in Inches Over Load at 0.2-on. Deflection Versus Deflection "N" in Inches for Bearing Plates of 36, 30, 24 and 12 in. in Diameter (Surface)

thousand pounds at any given deflection for the stronger subgrades. Consequently, some scattering of

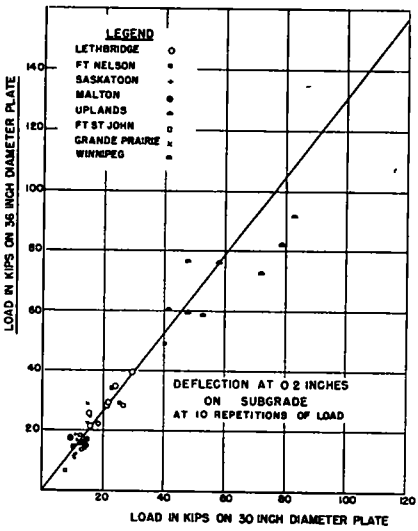

Figure 34. Load in Kips on 36-in. Diameter Plate Versus Load in Kips on 30-in. Diameter Plate at 0.2-in. Deflection

data about the best straight line through the graphs of Figs. 34 and 35 is to be expected. Figs. 34 and 35 contain data for Uplands airport (Ottawa), where the subgrade consists of 80 ft of clean sand, and for Fort Nelson where the subgrade is 3 to 5 ft of clean sand over clay. A quite different relationship would ordinarily be expected between load tests on bearing plates of different sizes on sand subgrade than on clay.

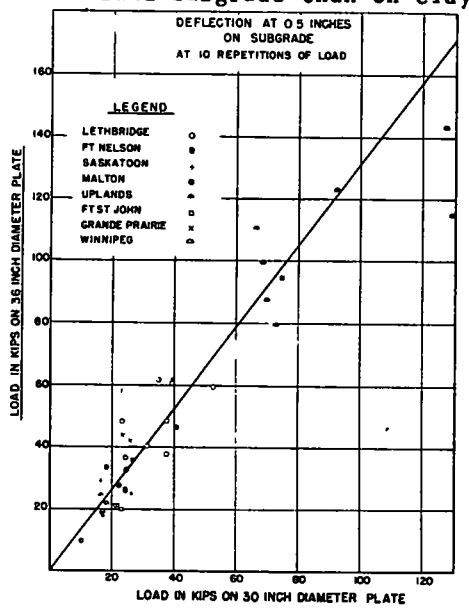

Figure 35. Load in Kips on 36-in. Diameter Plate Versus Load in Kips on 30-in. Diameter Plate at 0.5-in. Deflection

Since Figs. 34 and 35 indicate that the load test data for Uplands and Fort Nelson conform closely to those for airports with clay subgrades, it is evident that the upper layers of sand subgrade at Uplands and Fort Nelson contained sufficient inorganic or organic binder, or both, to make them behave in some respects as cohesive soils of high bearing capacity.

Information similar to that of Figs. 34 and 35 was developed at deflections of 0.05 and 0.1 to 0.7 in. in 0.1-in. increments for 12-in. versus 30-in. plates, 18-in. versus 30-in.

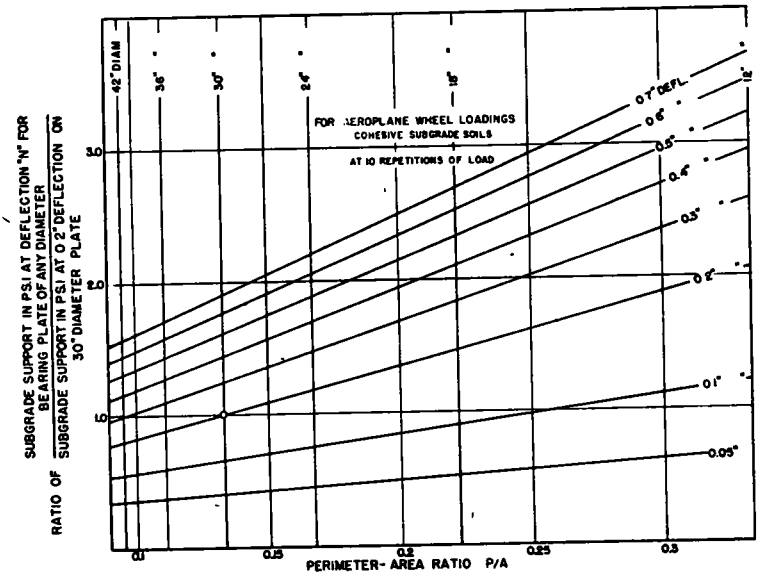

Figure 36. Ratio of Subgrade Support in PSI at Deflection Indicated for Bearing Plates of Any Diameter Over Subgrade Support in PSI at 0.2-in. Deflection on 30-in. Diameter Plate Versus Perimeter Area Ratio

plates, 24-in. versus 30-in. plates, 36-in. versus 30-in. plates, and 42-in. versus 30-in. plates. From this information, the ratio of the unit load supported on a plate of given size over the unit load supported on a 30-in. diameter plate could be readily determined for any required deflection. These ratios are indicated in Fig. 36 for deflections of 0.2 in. and 0.5 in. respectively. It will be noted that a very good straight line relationship has been obtained. Information for the

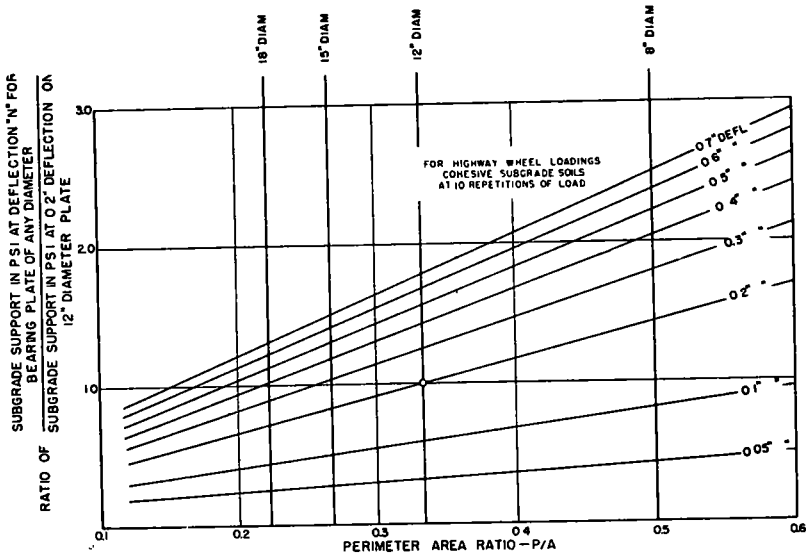

Figure 37. Ratio of Subgrade Support in PSI at Deflection "N" for Bearing Plate of Any Diameter Over Subgrade Support in PSI at 0.2-in. Deflection on 30-in. Diameter Plate Versus Perimeter Area Ratio

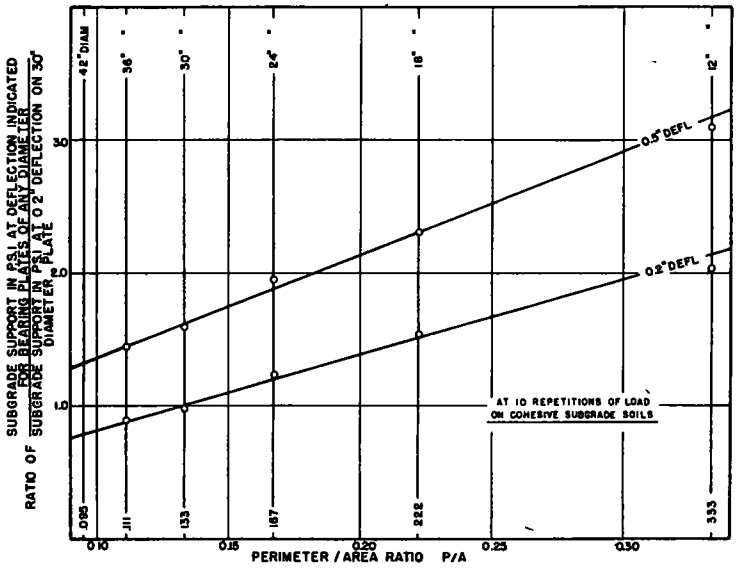

Figure 38. Ratio of Subgrade Support in PSI at Deflection "N" for Bearing Plate of Any Diameter Over Subgrade Support in PSI at 0.2-in. Deflection on 12-in. Diameter Plate Versus Perimeter Area Ratio

42-in. diameter plate has been omitted because it consisted of only five load tests made at a single airport and might not be representative. It should probably be added, however, that the data for these five tests would have placed the point for the 42-in. plate somewhat high in Fig. 36.

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. 25 for the 30-in. 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 (Fig. 36), it is a relatively simple matter to prepare the chart of Fig. 37. This chart is based upon the load carried by a 30-in. diameter plate at 0.2-in. deflection as a unity, or it could be considered to be based upon a unit load of 1 psi on a 30-in. diameter plate at 0.2-in. deflection.

The value of Fig. 37 lies in the fact that if the unit load supported on a 30-in. diameter plate at 0.2-in. deflection is known accurately, the unit load supported on a bearing plate for any other diameter over the range of 12 to 42 in. and probably somewhat beyond, can be calculated for any deflection between 0 and 0.7 in.

Figure 38 is similar to Fig. 37, with the exception that the diagram is based upon the load supported on a 12-in. diameter plate at 0.2-in. deflection as unity.

Figure 37 is intended for use for aircraft wheel loadings, since it covers the range of contact areas associated with the pneumatic tires on airplane wheels. For a similar reason, Fig. 38 is more readily applicable to highway wheel loadings. Diagrams similar to these could be very easily prepared, based on a ratio of unity, or a unit load of 1 psi, for any other combination of bearing plate size and deflection.

Again it is to be emphasized that Figs. 37 and 38, and any other diagrams based upon them, represent the average relationships derived from the analysis of a large number of load tests on bearing plates of different sizes, for what are considered to be representative cohe-

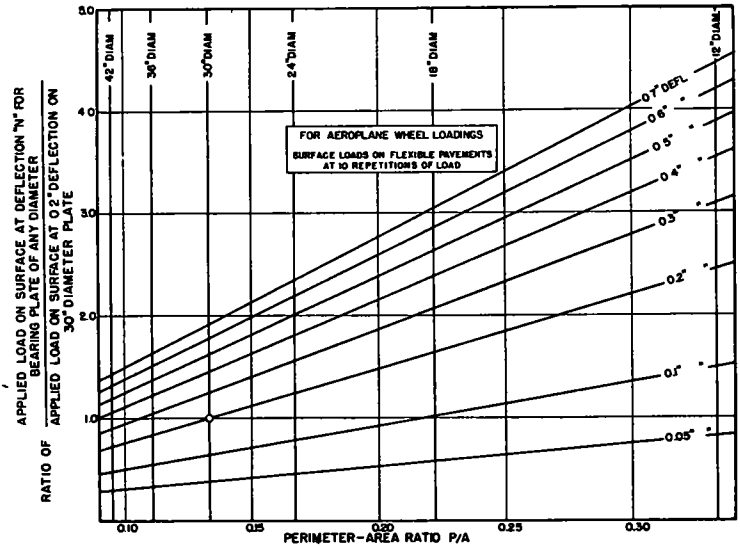

Figure 39. Ratio of Applied Load on Surface at Deflection "N" for Bearing Plates of Any Diameter Over Applied Load on Surface at 0.2-in. Deflection on 30-in. Diameter Plate Versus Perimeter Area Ratio

sive soils in Canada. Attempts to check these relationships with data from a few load tests may lead to disappointment. However, if the data from a relatively large number of load tests on cohesive subgrades are analyzed, reasonable agreement may be obtained, provided, of course, the load test procedure is identical with that outlined earlier in this paper.

From load tests made on the bituminous surfaces at the ten airports, information similar to that of Fig. 36 for subgrades was obtained for flexible pavements. When this information was combined with that of Fig. 33, the diagram of Fig. 39 was prepared. Figure 39 indicates that if the load supported by a flexible pavement on a 30-in. diameter plate at 0.2-in. deflection is accurately known, the load for any other bearing plate diameter over the range of 12 to 42 in. (and probably greater), and for a range of deflection from 0 to 0.7 in. can be calculated.

It may be that the diagram of Fig. 39 would vary somewhat for different bituminous surfaces, but a large amount of data, and a very costly investigation would be required to either establish or disprove this possibility.

Ratio of Unit Load on 12-in. Plate to that on a 30-in. Plate Figure 37 indicates that for deflections from about 0.15 in. to about 0.6 in., the unit load supported by a 12-in. diameter plate on cohesive subgrade soils is at least twice the unit load supported on a 30-in. plate at any given deflection. There may be exceptions to this observation in individual cases, but it is supported by not only our own investigation, but by the published data of Hubbard and Field(*8*), Campen and Smith(*9*),(*10*), Teller and Sutherland(*11*), and Middlebrooks and Bertram(*12*).

In the absence of load test information for a wide range of bearing plate diameters, this general rule is very useful when extrapolating or interpolating limited load test data to contact areas of different sizes.

For deflections greater than about 0.6 in., and smaller than

about 0.15 in., the unit load supported on a 12-in. plate on cohesive subgrade soils is somewhat less than twice the unit load supported on a 30-in. plate

For any deflection over the range 0 to 0.7 in., the actual ratio of load supported on a 12-in. plate to that supported on a 30-in. plate can be obtained by reference to Fig. 37 for cohesive soils.

For load tests on flexible pavements, Fig. 39 indicates that the ratio of the load supported on a 12-in. diameter plate versus that supported on a 30-in. diameter plate is somewhat greater than two. This ratio is approximately 2.45 over the range of deflection of about 0.2 to about 0.5 in. The exact ratio for any deflection can be obtained from Fig. 39.

YIELD POINT DEFLECTIONS FOR SUBGRADES AND FLEXIBLE PAVEMENTS

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. For loads up to the bearing capacity limit, the deflection reaches an equilibrium value not exceeding the yield point deflection. For loads greater than 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 on perimeter shear "*m*", developed pressure "*n*", and deflection "*d*" under each magnitude of load. From data for each of these variables, he calculates the soil resistance factors K_1 and K_2, where $K_1 = \frac{d}{n}$, and $K_2 = \frac{m}{n}$. When K_1 and K_2 values are plotted against 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. 37 and 39 are susceptible to analysis by Housel's method, and the results are presented in Figs. 40 and 41, respectively. Figure 40 indicates that the yield point occurs at a deflection of 0.26-in., where the K_2 curve reaches its maximum. This means that the average yield point deflection of subgrades for the ten airports is 0.26 in. Figure 41 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 in.

The very nearly identical yield point deflections indicated by Figs. 40 and 41 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.

Ordinarily, if the subgrade was the critical element in establishing the yield point of a flexible surface, the yield point deflection of the pavement could be expected to be somewhat higher than that of the subgrade, because of the compaction of the base and surface under load. However, for a well-compacted base and surface of 10 to 20 in. in thickness, there is evidence that this compaction may amount to only from 0.02 to 0.04 in.(*6*). That is, the yield point deflection of Fig. 41 for flexible surfaces would normally have been expected to be from 0.02 to 0.04 in. greater than the yield point deflection of Fig. 40 for subgrades.

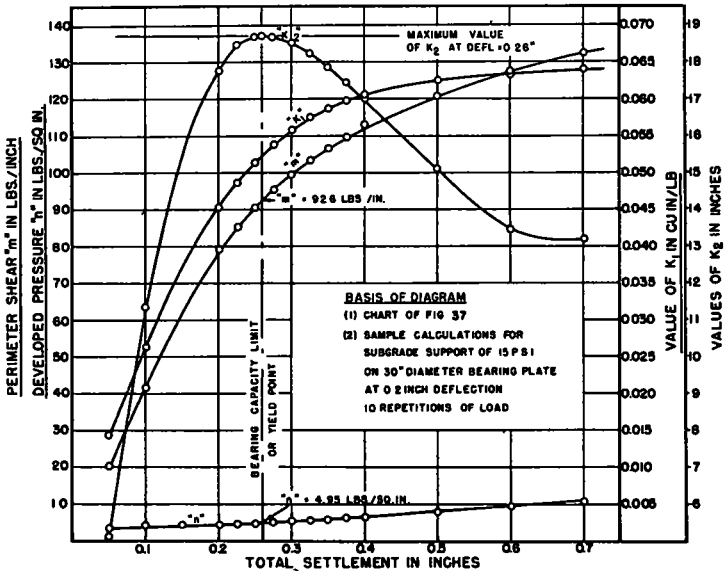

Figure 40. Yield Point Diagram for Cohesive Subgrades

Soils, however, are difficult materials to investigate from the point of view of stress and strain, and although the yield point deflections of Figs. 40 and 41 are in the reverse order to that ordinarily expected, the difference is so small that it could reasonably be attributed to experimental error. Those familiar with the difficulties normally encountered in the load testing of soils and flexible pavements are likely to be surprised that these yield point deflections should have turned out to be as close as they are. It should also be recalled that the information on which Figs. 40 and 41 are based, represents the average results of a large number of load tests on both subgrades and surfaces.

BASE COURSE LOAD SUPPORT VERSUS NATURE OF BASE COURSE MATERIAL

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 (Table 2).

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 (Fig. 42). 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 USED appears to have obtained data which point to a similar conclusion(6),(20).

It would be reasonable to expect that the load supporting value per unit thickness of a base course might vary with the composition of the base course material, its moisture content, and its density, all other factors being the same. Table 10 (see pg.89) lists the composition and density of the base courses at eight different airports. It will be observed that only at Regina, was the average density of

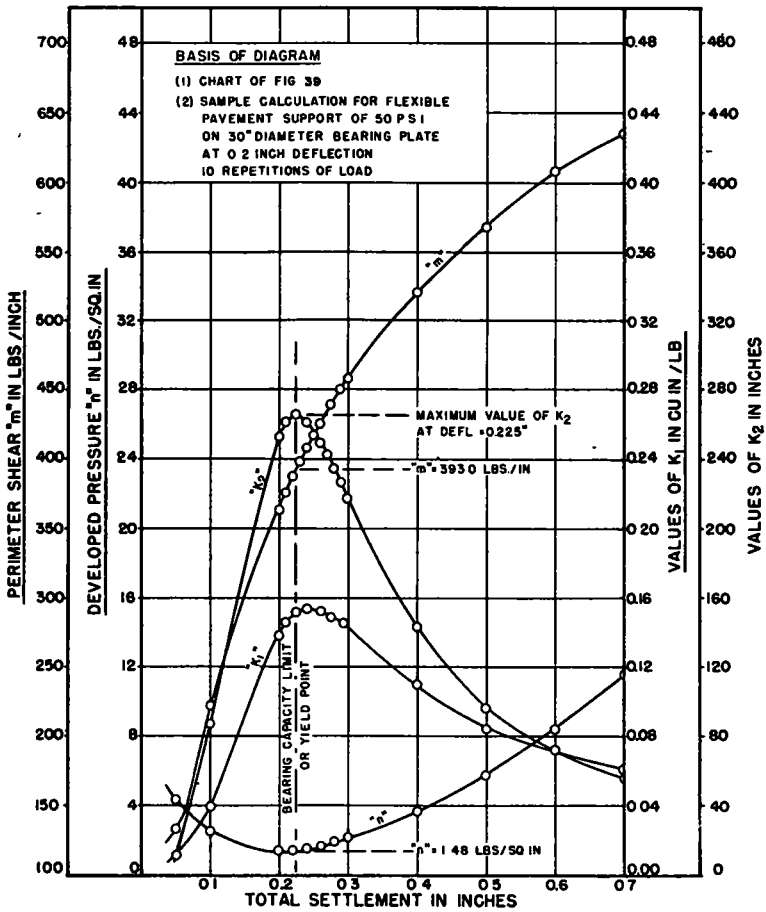

Figure 41. Yield Point Diagram for Flexible Pavements

the base course found to be greater than 100 percent of modified AASHO maximum density. It is significant that the base course at Regina indicated a much greater supporting value per inch of thickness than the base course at any of the other airports tested. There is no way of knowing at the present time, whether the greater supporting value per unit thickness of the base course at Regina is due entirely to its greater density, or partly to the higher density, and partly to its composition. More field load test data on base courses of different types placed at various densities and moisture contents, with all other factors remaining constant, would be required to provide further information on this problem.

The types of base course material and methods of base construction employed for the runways tested up to the present time, are probably quite representative of past and current airport and highway base course construction practice. When all the available information is considered, there is no positive evidence that for similar conditions of density and moisture content, all other factors being equal, that any one type of granular base material has a greater supporting value per unit of thickness than any other type. However, this matter merits considerable further study before a final conclusion can be stated.

In the selection of base course

material, it is well to remember that factors other than its load supporting capacity per unit thickness must usually be kept in mind. Most particularly, it must not itself undergo shear failure under the wheel loads to which it will be subjected. There was no evidence of base course shear failure at the ten airports tested in this investigation.

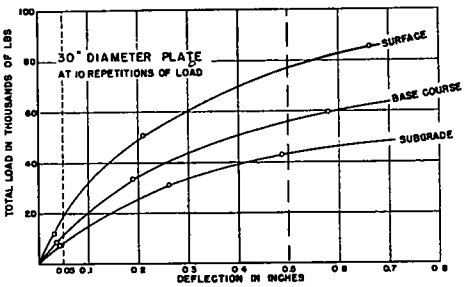

Figure 42. Load Versus Deflection Curves (Normal) for Subgrade, Base Course, and Surface

The question as to whether the subgrade soil itself is maintained in a stronger condition throughout the year under a porous gravel or macadam base, or under a dense well graded mechanically stabilized base course remains unsettled, and as will be seen later, this is a very important item of consideration.

BITUMINOUS SURFACE STRONGER THAN GRANULAR BASE

A study of the load deflection curves for subgrade, base course, and surface (Fig. 42) 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., 1 in. of thickness has the same load supporting capacity as about 1.5 in. of granular base. This ratio should probably not be applied for thicknesses of these types of pavement greater than about 4 in., unless justified by further load tests.

For well designed and constructed bituminous concrete, penetration macadam, and sheet asphalt, 1 in. of thickness of these types appears to have the same load carrying capacity as about 2.5 in. of granular base. Again, however, this ratio should probably not be applied for a thickness of these types of pavement greater than about 6 in., unless warranted by further investigation.

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.

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, cone bearing, and Housel penetrometer. If these relationships could be established, 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.

At all test locations, undisturbed samples were taken in cylinders 6 in. in diameter by 6 in. high (Fig. 43) at depths of 0 to 6 in. and 9 to 15 in. below the surface of the subgrade. They were immediately trimmed and sealed and then shipped to the laboratory for the determination of CBR values for

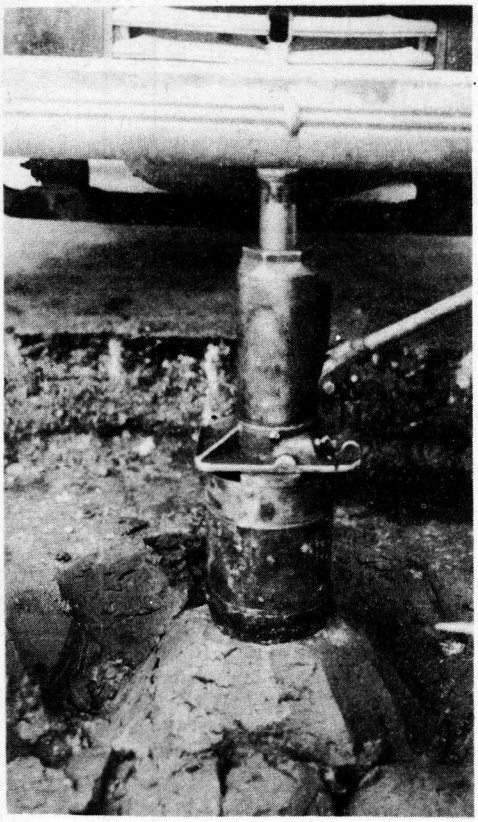

Figure 43. Equipment for Obtaining CBR Samples

both field and soaked conditions.

The field CBR values were obtained by testing in the "as recieved" condition. The soaked CBR values were determined after soaking the samples for 4 days according to standard procedure, and with required surcharge.

For the eight airports where the subgrade consists of cohesive soil, Table 7 lists the CBR values obtained for both field and soaked conditions. The CBR rating (soaked) of the subgrades for purposes of design as recommended in the U.S.A. would be approximately 2.3 to 4.5 for the eight airports. The field CBR values, on the other hand, varied with the condition of the subgrade soil under the pavement and the average ratings ranged from 3.9 to 13.3.

No relationships could be found between soaked CBR ratings and the corresponding load test data, but in Fig. 44 the field CBR values are plotted versus the measured subgrade support determined at each load test location. The field CBR value employed in each case was the average for subgrade depths of 0 to 6 and 9 to 15 in.

In arriving at the location of the best average line through the points of Fig. 44, the data could have been treated statistically without regard for the different airports to which they pertained. On this basis, however, the field

TABLE 7

CBR VALUES FOR BOTH FIELD AND SOAKED CONDITIONS, FOR EIGHT AIRPORTS WHERE THE SUBGRADE CONSISTS OF COHESIVE SOIL

	CBR Values					
	Field			Soaked		
	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

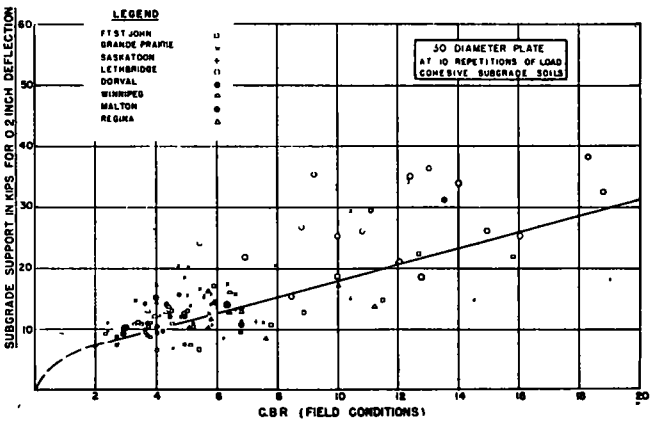

Figure 44. Subgrade Support Versus Field CBR

CBR values could lead to seriously over-estimating the actual subgrade support for some of the airports. For this reason, a different approach was adopted for the data of Figs. 44, 47, 50, and 60. The position of the best average line through the data of Fig. 44 was established on the basis that the subgrade support indicated by the curve for field CBR values must not exceed the actual subgrade support determined by a 30-in. diameter plate at 0.2-in. deflection, by more than about 10 percent for any one of the eight airports. That is, the location of the best average line through the data must not lead to over-estimating the true subgrade support by more than about 10 percent when the field CBR test is employed to measure subgrade bearing capacity directly.

It will be seen later (Figs. 94 and 96) that over-estimating the subgrade support by about 10 percent makes a difference of only 2 to 3 in. in the overall thickness of pavement required. It is believed that in the present state of our knowledge of flexible pavement design, the lack of accurate information concerning the other variables to be considered would lead to an error much greater than 2 to 3 in. when estimating the required thickness. Establishing the best average lines through the data of Figs. 44, 47, 50, 58, and 60 so that the subgrade support would be over-

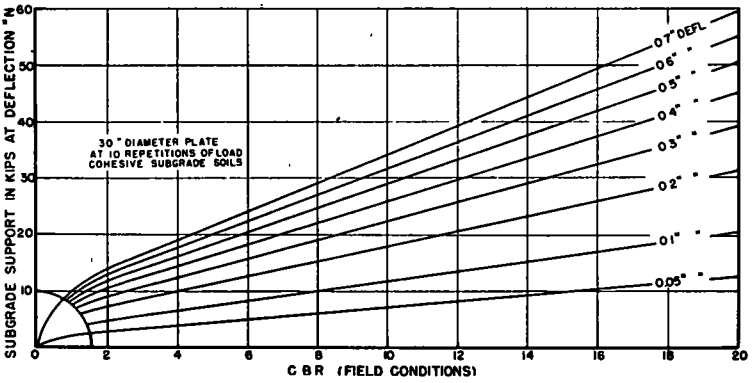

Figure 45. Subgrade Support in Kips at Deflection "N" Versus CBR (Field Condition)

estimated by not more than about 10 percent, when measured indirectly by the tests in question, therefore appears reasonable.

The scattering of points about the best average line through the data of Fig. 44 is due at least partly to the fact that the field CBR values are for the rectangular sampling area spaced approximately 12 ft from the location of the corresponding plate bearing test (Fig. 11) since even two load tests on a 30-in. diameter plate spaced 12 ft apart might vary by several thousand pounds. This scattering of points is probably also partly due to the normal experimental error to be expected when testing such a difficult material as soil.

In Fig. 45 relationships are indicated for CBR values versus subgrade support on a 30-in. diameter plate for a range of deflections from 0 to 0.7 in. Figure 45 results from a combination of the information contained in Figs. 44 and 25.

SUBGRADE LOAD TEST VERSUS CONE BEARING

Boyd(15), has described a cone bearing test that can be performed rapidly with very simple equipment (Fig. 46), 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 lb, in turn, and reading the penetration of the cone into the subgrade after each load in succession had 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.

In Fig. 47 cone bearing values are plotted against subgrade support on a 30-in. diameter plate at 0.2-in. 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 in. below the top of the subgrade, with the exception that where the value for the 0- to 6-in. 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-in. layers. This gave better agreement with the load test

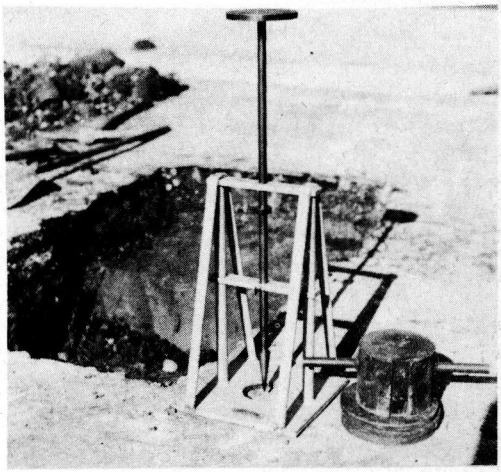

Figure 46. Equipment for Cone Bearing Test

data. It may be that a stronger 0- to 6-in. layer of subgrade has less influence on subgrade bearing capacity measured by plate bearing tests than underlying weak layers at the 6- to 12- and 12- to 18-in depth. There is also the possibility that the top of the subgrade may have been intruded by some base course material, which might result in higher cone bearing values for tests made on the surface of the subgrade than are actually representative for the 0- to 6-in. layer. Probably the cone bearing test should be made at the depths of 3, 9, and 15 in. for the 0- to 6-, 6- to 12-, and 12- to 18-in. layers, respectively, to obtain more representative results for each layer. In this case the most suitable cone

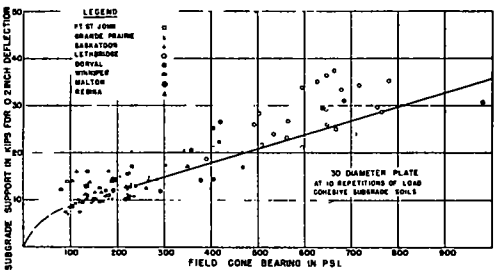

Fig. 47 Subgrade Support Versus Field Cone Bearing

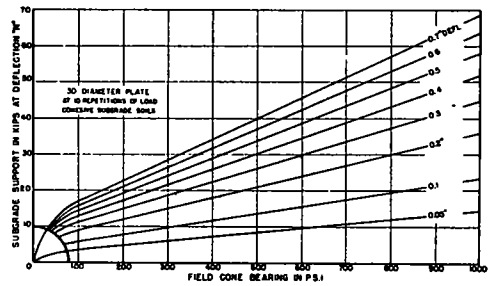

Figure 48. Subgrade Support in Kips at Deflection "N" Versus Field Cone Bearing in PSI

bearing value to be employed would probably be the average for the three layers at each test location.

The position of the best average line through the data of Fig. 47 was established on the basis previously described for Fig. 44. Relationships for cone bearing values versus subgrade support on a 30-in. diameter plate for a range of deflection between 0 and 0.7 in. are indicated in Fig.48, which is obtained by combining the information of Figs. 47 and 25.

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(*16*) (Fig. 49) consists of a sharpened 1.25-in. diameter standard pipe, which with accessories weighs exactly 20 lb, exclusive of the driving weight, which also weighs exactly 20 lb. Stops on the barrel of the 1.25-in. pipe control the height of drop of the driving weight to exactly 34 in. The test consists of determining the number of blows of the 20-lb driving weight falling exactly 34 in. required to drive the sharpened pipe 6 in. into the soil. A cardboard strip firmly attached to the barrel of the pipe (Fig. 49) is marked with a pencil at the beginning of the test and after the penetration of each blow.

Figure 50 is a graph of Housel penetrometer values versus subgrade support on a 30-in. diameter plate at 0.2-in. 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 in. below the surface of the subgrade.

By combining the information contained in Figs. 50 and 25, the relationships of Fig. 51 are determined for Housel penetrometer values versus actual subgrade support on a 30-in. plate for deflections between 0 and 0.7 in.

SUBGRADE LOAD TEST VERSUS TRIAXIAL COMPRESSION TEST

Through the use of Figs. 45, 48, and 51, field CBR, cone bearing, and Housel penetrometer tests can each be employed in the field to determine indirectly subgrade

bearing values for a 30-in. plate for deflections of from 0 to 0.7 in.

It was also desired to develop a laboratory test 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.

A triaxial compression test differs from an ordinary compression test in that provision is made for controlled lateral support while the specimen is subjected to vertical compression. A diagram of the triaxial compression equipment employed to obtain the data reported on here is given in Fig. 52. It consists of a lucite cylinder to which two metal end-plates are fitted by means of watertight and airtight steel joints. The soil test specimen is cut from the large undisturbed sample sent from the field, inserted in a rubber sleeve between two porous stones, and placed between the base and piston which exert the vertical load. By means of connections through the porous stones, the soil sample can be subjected to vacuum or water pressure, but neither was employed for the data reported here. Water or air can be pumped into the lucite cylinder to provide the magnitude of lateral support required. For this investigation, constant lateral pressures of 0, 15, and 30 lb per sq in. respectively, were used. Consequently, three test specimens were required from each undisturbed sample sent in from the field.

Figure 53, commonly known as a Mohr diagram, indicates the nature of the information obtained from a triaxial compression test. The applied lateral pressure and the corresponding vertical pressure which caused failure, are marked off on the horizontal axis for each

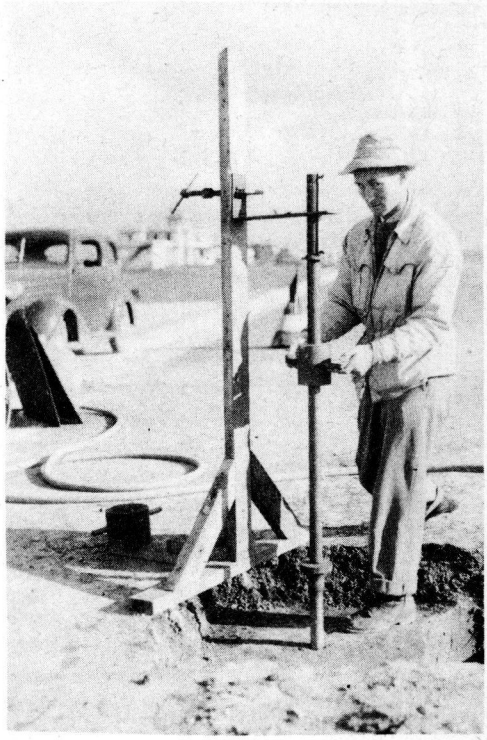

Figure 49. Equipment for Housel Penetrometer Test

of the three test specimens. Using the difference between the vertical and lateral pressure in the case of each specimen as diameter, semi-circles are described as shown. The tangent which is common to the three semi-circles is drawn, and is produced to intersect the vertical axis. The magnitude of the intercept on the vertical axis is a measure of the value of the cohesion c from the Coulomb equation $s = c + n \tan \phi$, while the angle between the common tangent and the horizontal is the angle of internal friction ϕ.

The common tangent is generally known as the Mohr rupture line. All semi-circles which are tangent to or below the Mohr rupture line, represent equilibrium or stable relationships, respectively, between the vertical and lateral pressures marked off by the semi-circles on the horizontal axis.

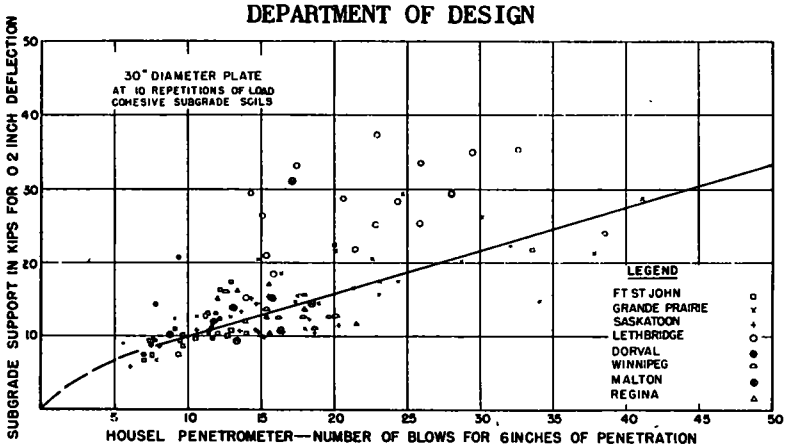

Figure 50. Subgrade Support Versus Housel Penetrometer

If the horizontal 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 CBR values, for example, are for the cone bearing or CBR tests, respectively. From Fig. 53 it is obvious that this representative value for the triaxial compression test must come from the cohesion c, the angle of internal friction Ø, the lateral pressure L, the vertical pressure V, or from some combination of two or more of these variables.

A relationship might have been expected between the load test data and the results of unconfined compression tests. However, when the vertical pressures (ultimate or at some definite deflection) for a lateral support of zero were plotted versus load test data (30-in. plate at 0.2-in. deflection), the points were so widely scattered on the graph paper that it appeared evident that no relationship could be developed between load test data and the results of unconfined compression tests made with the triaxial compression equipment. At least, this conclusion is justified for the five airports with cohesive subgrade soils for which data are available. Nor could any relationship be found between plate bearing

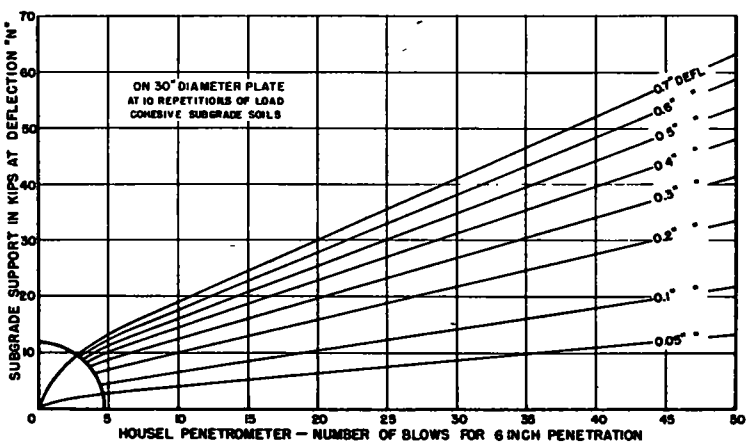

Figure 51. Subgrade Support in Kips at Deflection "N" Versus Housel Penetrometer

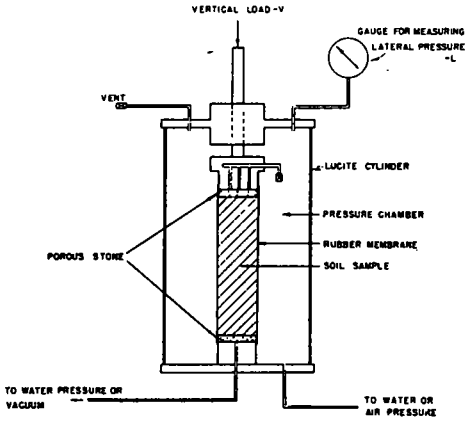

Figure 52. Sketch of Apparatus for Triaxial Compression Test

test results and the vertical pressure for any other than lateral pressure in the triaxial compression test.

Some other approach was therefore required and Fig. 54 illustrates the geometrical and trigonometrical relationships that are employed for the development which follows this section. The two Mohr rupture lines in Fig. 54, 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.

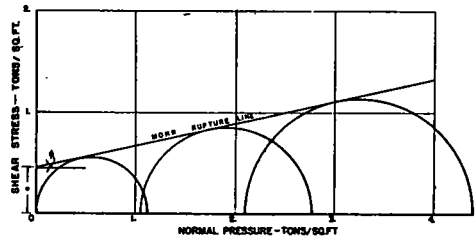

Figure 53. Typical Mohr Diagram for Triaxial Compression Test

The equations required for the following paragraphs are listed in Fig. 54. Of particular importance is the equation:

$$a = 2c \tan\left(45 - \frac{\phi}{2}\right)$$

wherein a is the value of lateral pressure L for the Mohr circle for which the vertical pressure V is zero, and

$$b = \frac{V_o - L_o}{V_o}$$

wherein the values V_o and L_o are vertical and lateral pressures for Line B, which is parallel to Line A

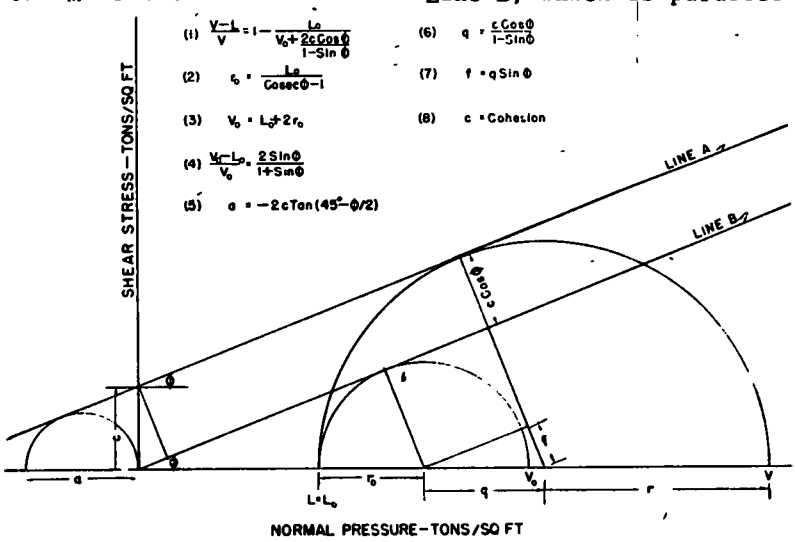

Figure 54. Diagram Illustrating Certain Geometrical Relationships for Triaxial Compression Test Data

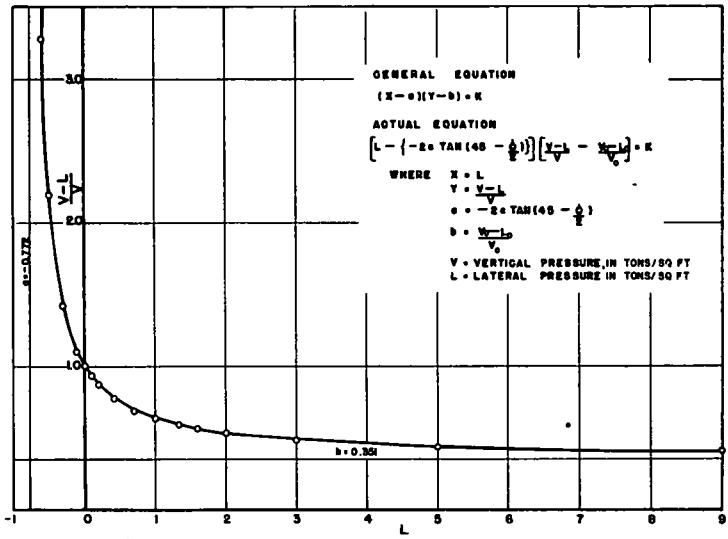

Figure 55. Ratio of $\frac{V - L}{V}$ Versus Lateral Pressure L for Triaxial Compression Test

but passes through the origin.

When studying the relationships between the different variables of the Mohr diagram (Fig. 53) it was found that a rectangular hyperbola resulted if $\frac{(V - L)}{V}$ was plotted against L (Fig. 55). This curve conformed to the general equation

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

where each symbol has the significance indicated in Fig. 55, and K is a constant.

It was also found that a rectangular hyperbola resulted when $\frac{(V - L)}{V}$ was plotted against V (Fig. 56) and this curve was represented by the general equation

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

where each symbol has the significance indicated in Fig. 56 and K is a constant.

It is obvious that there is nothing significant about the curves of Figs. 55 and 56. However, when $\log \frac{V - L}{V}$ is plotted against log L, the reverse curve graph of Fig. 57 is obtained.

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

It was thought that the slope of the reverse curve of Fig. 57 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 is referred to in the balance of this paper 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. 57 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

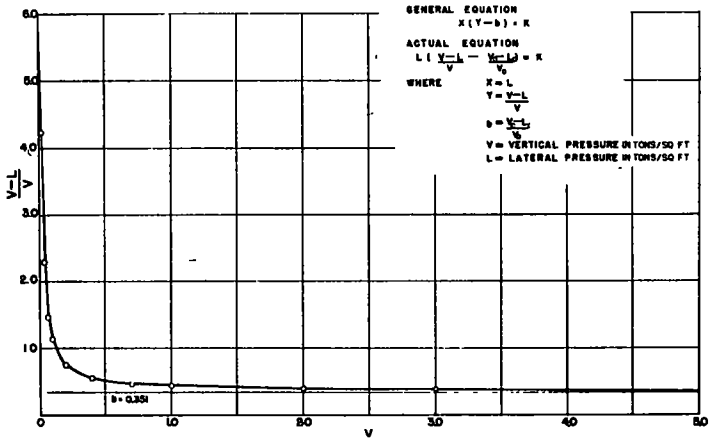

Figure 56. Ratio of $\frac{V-L}{V}$ Versus Vertical Pressure V for Triaxial Compression Test

(slope factor m) and of lateral pressure L at the point of inflection is given on Fig. 57 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. Figure 58 is a graph of the values of slope factor m versus load test results on a 30-in. plate at 0.2-in. deflection for the six airports with cohesive subgrade soils for which triaxial compression test data were obtained. It is apparent that a relationship exists, and a best

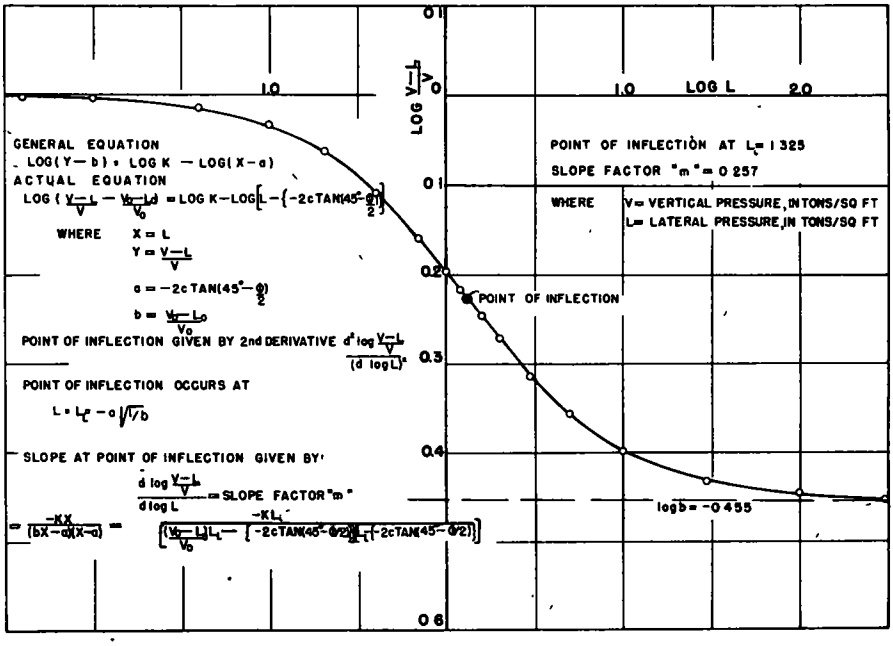

Figure 57. Log of Ratio $\frac{V-L}{V}$ Versus Log L for Triaxial Compression Test

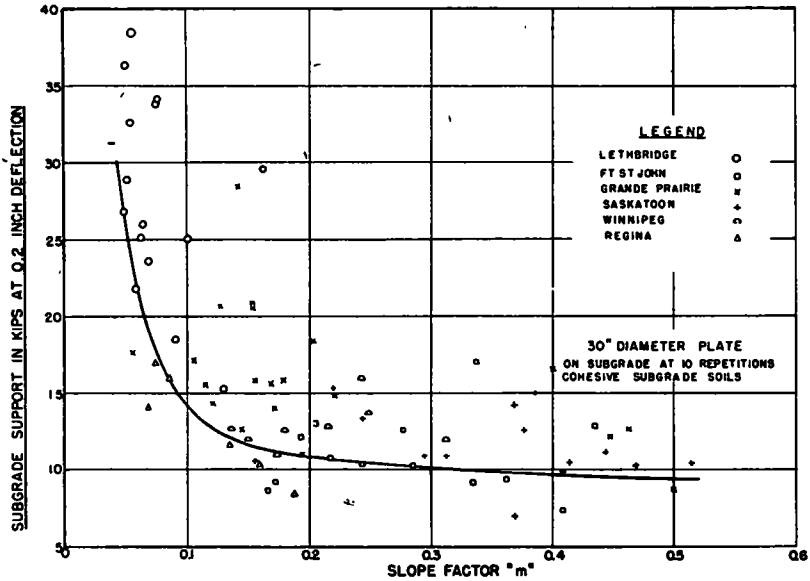

Figure 58. Subgrade Support in Kips at 0.2-in. Deflection Versus Slope Factor m

average line can be drawn through the points 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 Ø. The relationship between slope factor m and angle of internal friction Ø is shown in Fig. 59.

Consideration of the information in Figs. 58 and 59 implies that a relationship should exist between angle of internal friction Ø and subgrade support on a 30-in. plate at 0.2-in. deflection (Fig. 60).

An attempt was made to establish the best average line through the data of Fig. 60 on the usual basis that the subgrade bearing capacity indicated by the Ø values curve would not exceed the true subgrade support on a 30-in. plate at 0.2-in. deflection by more than about 10 percent 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 percent.

There is 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-in. plate at 0.2-in. deflection from Fig. 58) when this information can be determined with identical accuracy from Fig. 60 based upon the angle of internal friction Ø as read directly from the Mohr diagram.

Figure 61 indicates the relationships for angle of internal friction Ø versus subgrade support on a 30-in. plate for deflections from 0 to 0.7 in.

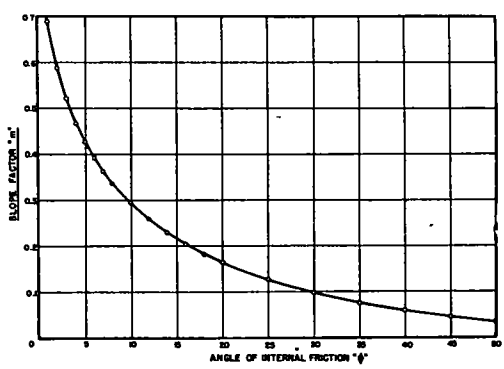

Figure 59. Slope Factor m Versus Angle of Internal Friction

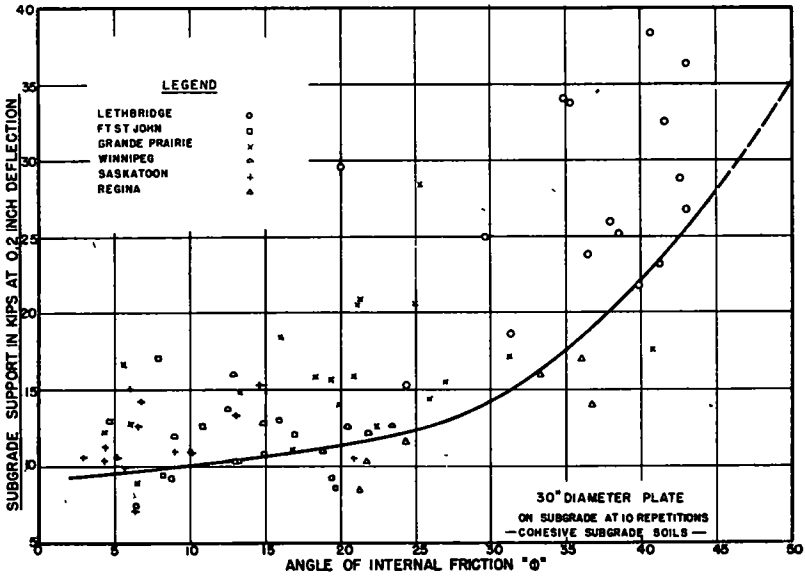

Figure 60. Subgrade Support in Kips at 0.2-in. Deflection Versus Angle of Internal Friction

If L_1 represents the lateral pressure at the point of inflection (Fig. 57) and V_i is the corresponding vertical pressure, it might be expected that relationships should exist between subgrade support (on a 30-in. plate at 0.2-in. deflection) and V_i, $(V_1 - L_i)$, L_i, or $\frac{(V_i - L_i)}{V_i}$

While the relationship might be particularly anticipated between subgrade support on a 30-in. plate and V_i, the scattering of data on the graph paper indicated that none appears to exist. This is equally true when load test data are plotted against $(V_i - L_i)$.

A reasonable graph is obtained when L_i is plotted against subgrade support on a 30-in. plate (Fig. 62) but because of the sharpness of the curve and the flatness of the lower portion it seems less satisfactory

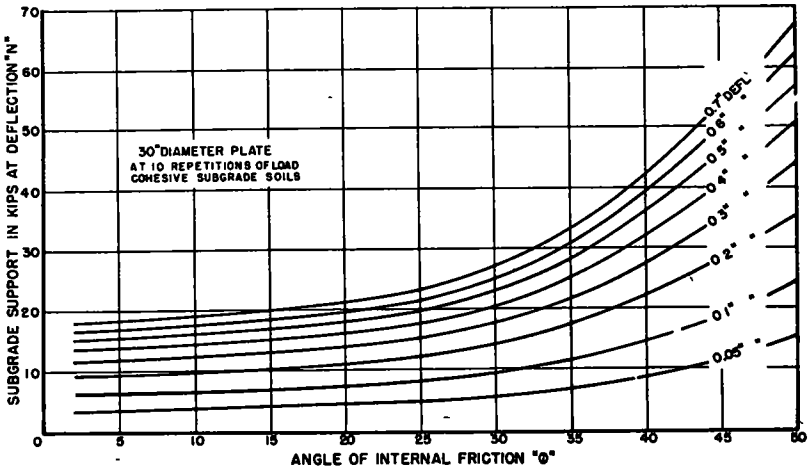

Figure 61. Subgrade Support in Kips at Deflection "N" Versus Angle of Internal Friction

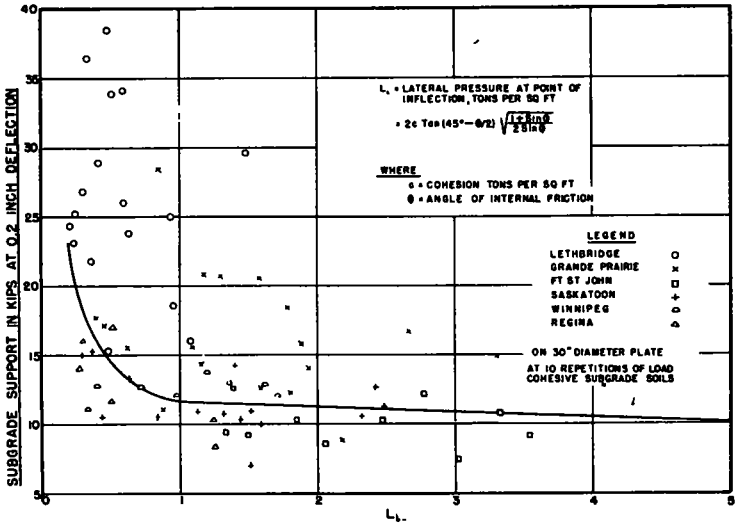

Figure 62. Subgrade Support in Kips at 0.2-in. Deflection Versus L_i for Triaxial Compression Test

as a basis for design than the relationship between ϕ and load test data of Fig. 60. On the other hand, the graph of Fig. 62 has the advantage that L_1 includes values of both c and ϕ from any Mohr diagram.

A very good relationship exists between subgrade support on a 30-in. diameter plate and $\frac{(V_i - L_1)}{V_i}$ (Fig. 63). However, in any ratio such as $\frac{V_i}{L_1}$, or $\frac{(V_i - L_i)}{V_1}$, etc., the cohesion term c cancels from both numerator and denominator, and the expression is seen to be dependent on ϕ only. Therefore, $\frac{(V_i - L_i)}{V_i}$ is independent of cohesion c and is a function of ϕ only. Consequently, there is no advantage in determining the value of $\frac{(V_1 - L_1)}{V_i}$ since a similar relationship with respect to load test values can be determined from ϕ (Fig. 60).

CONE BEARING, HOUSEL PENETROMETER, FIELD CBR, AND TRIAXIAL COMPRESSION TESTS COMPARED

In Table 8, a comparison is made between the load test data obtained indirectly from the best average line through the field CBR, cone bearing, Housel penetrometer, and triaxial compression test data of Figs. 44, 47, 50, and 60, 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 (over-estimate of subgrade support must not exceed about 10 percent for any airport), it is obvious that the locations of the best average lines would fit the data for some airports better than for others. Table 8 indicates for each airport, the deviation between load test information obtained indirectly from the correlation curves

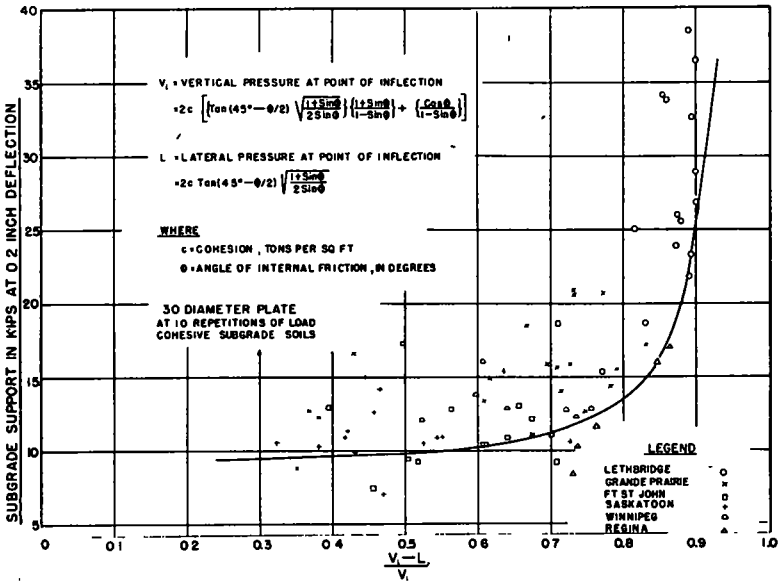

Figure 63. Subgrade Support in Kips at 0.2-in. Deflection Versus $\frac{V_i - L_i}{V_i}$ for Triaxial Compression Test

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 over-estimated the subgrade support for that airport by the amount of the difference between the percentage given and 100 percent. Similarly, the subgrade support has been underestimated according to the best average line, wherever the percentage shown in Table 8 is less than 100.

In general, reasonably good agreement is indicated by Table 7,

TABLE 8

RATIO OF LOAD TEST VALUES GIVEN BY BEST AVERAGE LINE THROUGH DATA FOR CONE BEARING, HOUSEL PENETROMETER, FIELD CBR, 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.

Airport	Cone Bearing	Housel Penetrometer	Field CBR	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

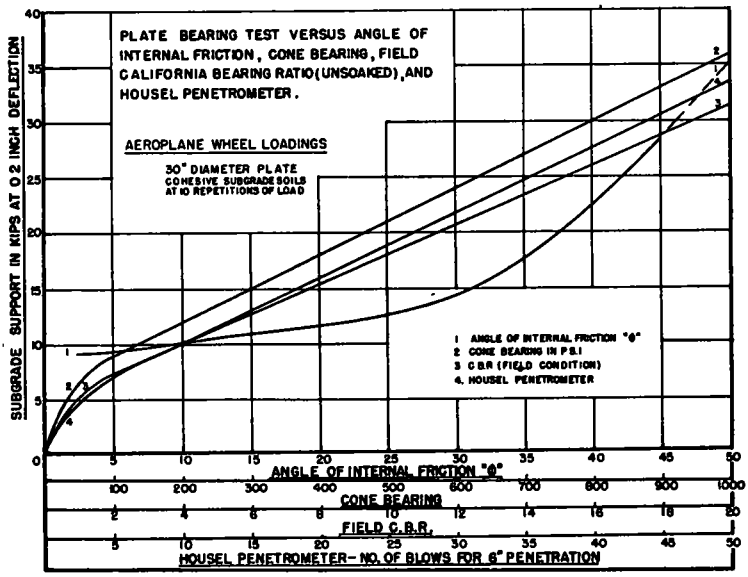

Figure 64. Plate Bearing Test Versus Angle of Internal Friction, Cone Bearing, Field California Bearing Ratio (Unsoaked), and Housel Penetrometer - (Airplane Wheel Loadings)

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 30 percent, and in only five cases is it greater than 20 percent.

From the right-hand column of Table 8, it will be seen that when the results of the four tests are averaged, the actual load test data are approximately within 10 percent 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 8 are averaged, the results are within 10 percent of the actual load test data for all airports except Lethbridge.

From the point of view of simplicity of test equipment and test procedure, and of overall accuracy of results, the cone bearing test appears to be the best of these four tests. The cone bearing test is performed with very simple equipment which can be easily handled by one man. The amount of load required is small and is part of the equipment. The device for marking the degree of penetration under each load is built into the apparatus. The test equipment is quite light, self-contained, can be set up is a few moments, and each test completed in about five or six minutes. Several tests should be made on each layer of subgrade to provide a satisfactory overall average value for the layer.

The Housel penetrometer is also a very rapid test, which can be made with simple apparatus. Table 8 indicates that its accuracy as a measure of subgrade support is almost equal to that of the cone bearing.

The CBR test has been widely used in the U.S.A. When used as a field test, however, it is more complicated and time-consuming than the cone bearing or Housel penetrometer test. It requires a deflection beam and deflection gauge to measure the penetration of the

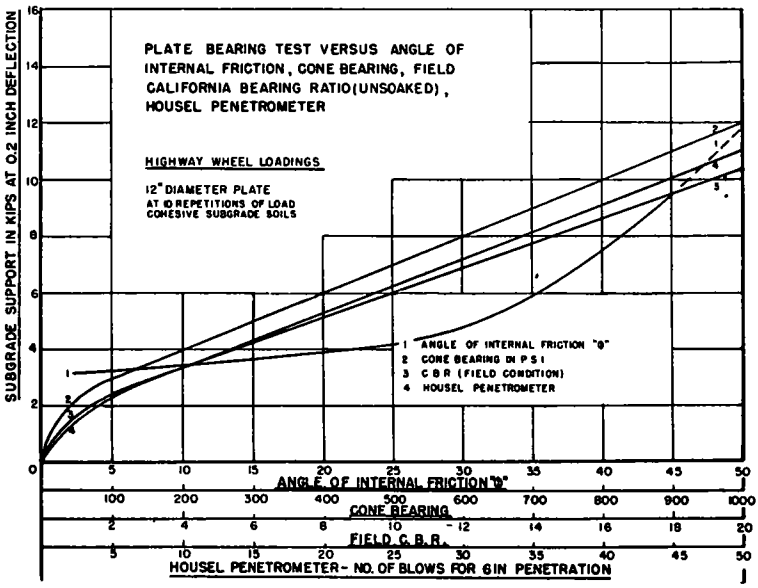

Figure 65. Plate Bearing Test Versus Angle of Internal Friction, Cone Bearing, Field California Bearing Ratio (Unsoaked), and Housel Penetrometer - (Highway Wheel Loadings)

piston, and these must be set up to be independent of the rest of the equipment. A source of load weighing many hundreds of pounds is required to jack against in order to obtain the required penetration of the piston into the subgrade. The rate of loading the piston must be, or should be controlled to give the rate of penetration specified. An hydraulic gauge, spring gauge, or proving ring is needed to indicate the magnitude of the load being applied. Some time is therefore required to set up the equipment for each test made in the field. Like the cone bearing equipment, several tests should usually be made on each subgrade layer, each of which requires a new set-up of the equipment.

An alternative to making the CBR test in the field is to obtain undisturbed samples of the subgrade over the depths required and ship them to the laboratory for test.

In Fig. 64 the cone bearing, field CBR, Housel penetrometer, and Ø (from the triaxial compression test) values are plotted against subgrade support on a 30-in. diameter plate at 0.2-in. deflection for cohesive subgrade soils. Consequently, Fig. 64 combines into one graph, the information provided by the best average lines through the data of Figs. 44, 47, 50, and 60. Because it refers to a 30-in. plate, Fig. 64 may be considered more applicable to airplane wheel loading.

In Fig. 65, information similar to that of Fig. 64 is given relative to a 12-in. diameter plate at 0.2-in. deflection. A 12-in. plate has a contact area similar to that of many highway wheel loadings.

Figure 66 consists of a chart which provides relationships between plate bearing tests on a 30-in. plate at deflections of 0.2 and 0.5 in., plate bearing tests on a 12-in. plate at deflections of 0.2 and 0.5 in., subgrade modulus k determined with a 30-in. plate, and values of cone bearing, field CBR, Housel penetrometer, and triaxial compression tests, all with reference to 10 repetitions of load on cohesive subgrade soils.

One of the principal advantages

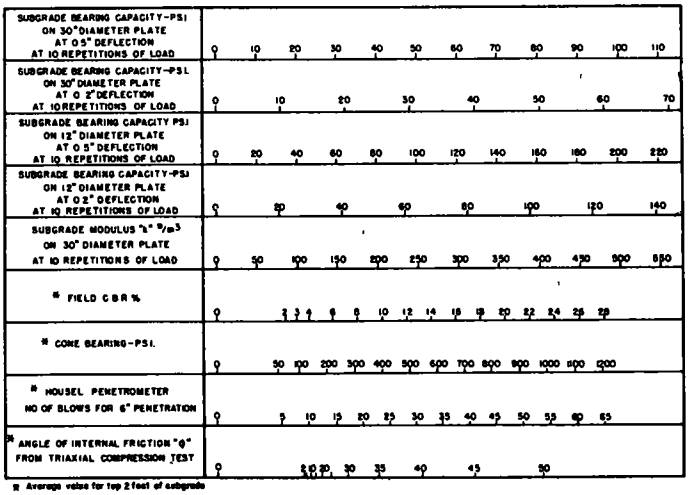

Figure 66. Relationships Between the Measurements of Subgrade Bearing Capacity Indicated by Various Test Methods (for Cohesive Subgrade Soils)

in having the four separate tests, cone bearing, Housel penetrometer, field CBR, and triaxial compression, all correlated with load test results, lies in the fact that one test can be used as a check against another to reduce the possibility of error. It has already been pointed out that when cone bearing and Housel penetrometer tests are combined (Table 8), the margin of error may not exceed 10 percent in general.

Another principal advantage is due to the fact that there are many locations where load testing is out of the question owing to inaccessibility, or because the subgrade soil is in a different condition than will occur after paving. New locations where no paving exists fall into the latter category. Load test results in these new areas are useless because the soil moisture after paving, and therefore the subgrade support, may be quite different from that at the time the load tests were made.

Samples of the soil from these areas may be taken into the laboratory however, compacted to the moisture content and density which experience elsewhere has indicated they will attain (Figs. 3, 4, 5, 6, and 7) and cone bearing, CBR, triaxial compression, and Housel penetrometer tests made on them in this condition in the laboratory. From these latter values the probable subgrade bearing capacity for the soil in its ultimate condition under the pavement can be read from the graphs of Figs. 64, 65, or 66.

A few cautionary comments are in order in connection with the use of cone bearing, field CBR, Housel penetrometer, or triaxial compression test results for indirectly obtaining bearing capacity values for cohesive subgrade soils:

1. The subgrade should be in the condition of moisture and density ultimately anticipated under the pavement.

2. Tests should be made throughout the top 18 in. of the subgrade for highways, and throughout the top 2 ft of the subgrade for airport runways. Average values should be determined for each 6-in. layer throughout this depth.

3. Care must be taken that

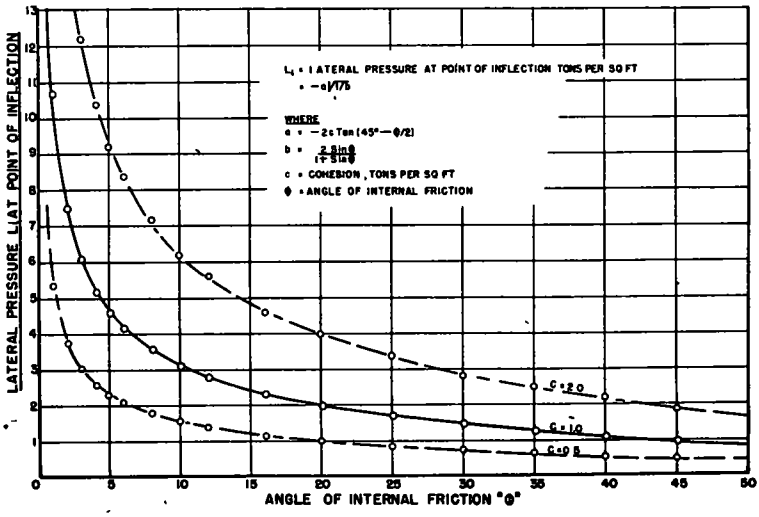

Figure 67. Lateral Pressure Versus Angle of Internal Friction for Various Values of Cohesion for Triaxial Compression Test

the results of any one or more of the four tests are reasonably representative of the soil over the full depths actively affected by the wheel loads of the anticipated traffic. Soft soil below a depth of 3 ft, for example, would not be detected by cone bearing, field CBR, Housel penetrometer, or triaxial compression tests made on layers to a depth of 2 ft, but could be the cause of later failure. For highway subgrades, the subgrade soil should be explored by borings to a depth of at least 4 ft. For subgrades for airport runways for heavy planes, these borings should be made to a depth of not less than 10 ft, and preferably to 20 ft. If a soft layer of soil is encountered over these depths, some modification of the bearing capacity established indirectly by the tests on the top 2 ft of the subgrade would be indicated.

4. One advantage of the load test for determining the bearing capacity of a cohesive subgrade in which a soft layer of soil may occur, is that it is probably influenced by a soft subgrade layer at any depth affected by an airplane or truck wheel load, whereas the cone bearing, field CBR, Housel penetrometer, or triaxial compression tests are not influenced by the character of the soil more than a few inches away. For cohesive soils which may have a soft layer within the depth affected by the wheel loads of traffic, values of subgrade bearing capacity determined indirectly by the four tests considered in this section should probably be modified downward.

5. Let it be emphasized again that no one or more of these four tests should be employed to determine subgrade bearing capacity unless the subgrade is in the most critical condition of moisture and density ultimately anticipated under the pavement, or unless they have been made on samples of the subgrade prepared at these conditions of moisture content and density in the laboratory.

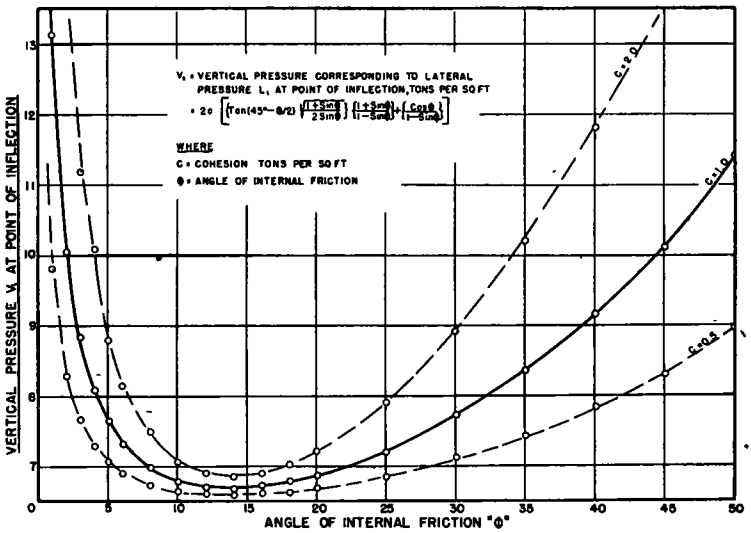

Figure 68. Vertical Pressure Versus Angle of Internal Friction for Various Values of Cohesion for Triaxial Compression Test

BITUMINOUS MIXTURE DESIGN BY THE TRIAXIAL COMPRESSION TEST

A further development of the information from the triaxial compression test is its application to the design of bituminous paving mixtures.

Figures 67, 68, 69, and 70 illustrate relationships involving L_i and V_1, the lateral and vertical pressures respectively, corresponding to the point of inflection in Fig. 57. Figure 67 shows the relationship between L_i and ϕ for several values of cohesion c. In Fig. 68 the graph of V_1 versus ϕ is given for three values of cohesion c.

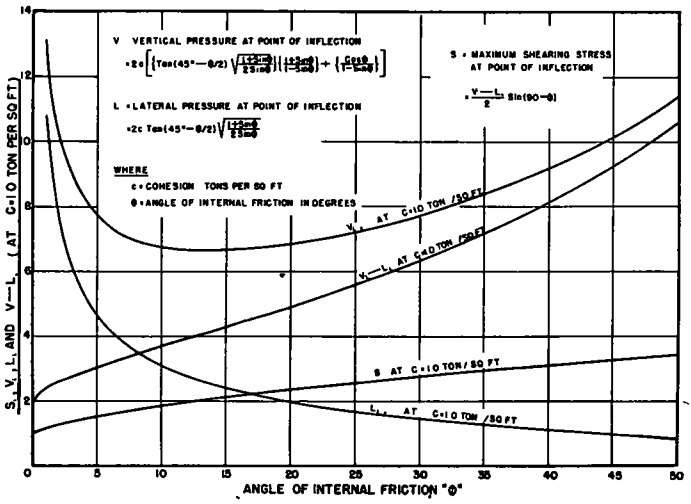

Figure. 69. S_i, V_i, L_i and $(V_i - L_i)$ Versus Angle of Internal Friction When Cohesion c = 1.0 ton per sq ft

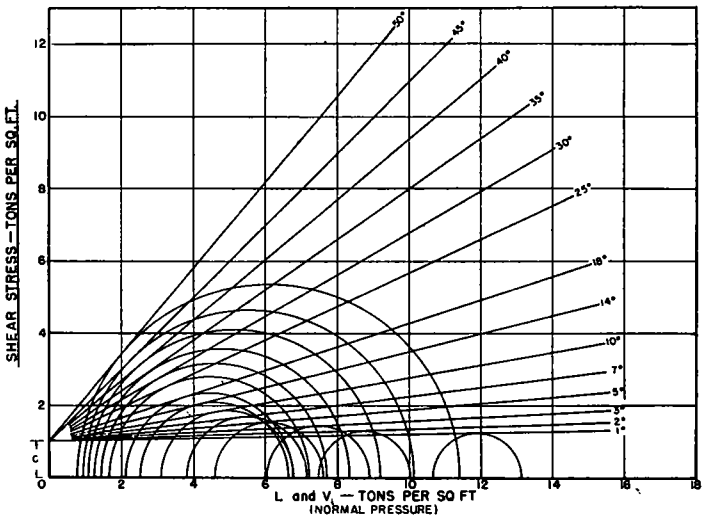

Figure 70. Mohr Diagram Illustrating Influence of Variation in Angle of Internal Friction on Values of L_i and V_i When Cohesion is Constant

Like L_1, the value of V_i depends upon both ϕ and c.

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

$$V_1 = 2c\left[\frac{1 + \sin\phi + \sqrt{2\sin\phi}\sqrt{1+\sin\phi}}{\sqrt{2\sin\phi}\ \sqrt{1-\sin\phi}}\right.$$

In Fig. 69, S_i, L_1, V_1, and $V_i - L_1$ are plotted against ϕ for a value of cohesion c equal to unity in each case. S_i is the maximum shear stress at the point of inflection.

Figures 67 and 69 indicate that when c is constant, for each value of L_1 there is only one corresponding value of ϕ. Figures 68 and 69, on the other hand, show that when c is constant, for each value of V_i there are two corresponging 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_i 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 a value of 13.7 deg after which V_i increases as ϕ increases. The reason for this can be readily seen by reference to Fig. 70, which is a Mohr diagram in terms of L_i and V_i, with c constant but ϕ variable.

Cohesion c in Fig. 70 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 deg. When $\phi \longrightarrow$ zero, both V_i and L_i are infinitely great and are therefore off the diagram to the right. For values of ϕ from 0 to 13.7 deg, both V_i and L_i steadily decrease as ϕ increases. For all values of ϕ greater than 13.7 deg, L_i continues to decrease, while V_i, on the other hand, begins and continues to increase as ϕ increases from 13.7 to 90 deg. Consequently, for any given value of cohesion c, the lowest value of V_1 occurs at an angle of 13.7 deg, and V_i increases as ϕ decreases toward 0 or increases toward 90 deg from this critical value of 13.7 deg.

Figure 70 demonstrates that for a constant value of c, the value of $(V_i - L_i)$ decreases as ϕ decreases, and vice versa. It can be similarly shown that for a constant value of ϕ,

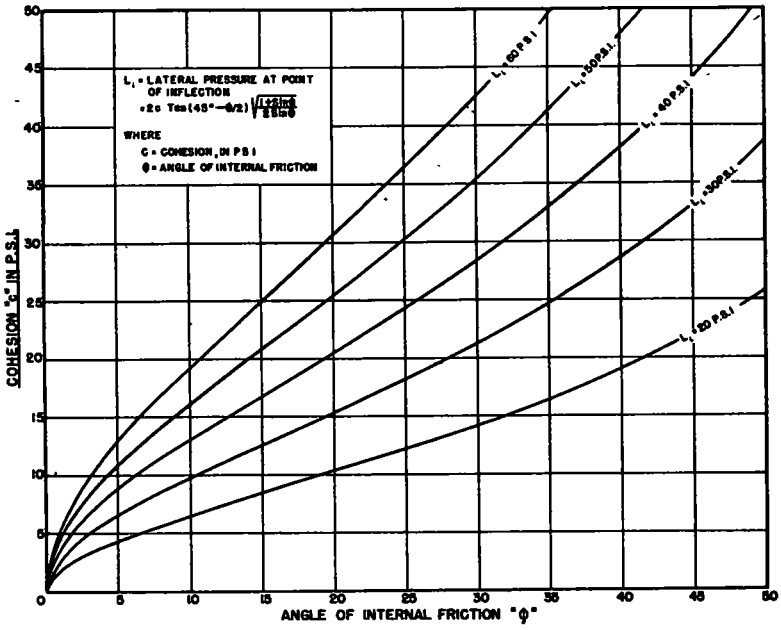

Figure 71. Relationships Between Cohesion, Angle of Internal Friction and Lateral Pressure at the Point of Inflection for Triaxial Compression Test

$(V_i - L_i)$ decreases as c decreases, and vice versa. Consequently, $(V_i - L_i)$ values represent a measure of the stability of a cohesive material being tested by triaxial compression.

Figure 69 indicates that $(V_i - L_i)$ increases as ϕ increases, when c is constant, but a point of inflection occurs when ϕ is equal to 13.7 deg.

Figures 71, 72, 73, and 74, are graphs of cohesion c versus angle of internal friction ϕ, for values of L_i, V_i, $(V_i - L_i)$, and S_i respectively. Figure 71 indicates that the curve for each value of L_i rises as c and ϕ are both increased, and a point of inflection occurs when ϕ = 13.7 deg.

Figure 72 shows that the curve for each value of V_i rises as c and ϕ increase, over the range of ϕ between 0 and 13.7 deg. Thereafter the curve falls as ϕ increases beyond 13.7 deg.

Figure 73 indicates that the curve for any given value of $(V_i - L_i)$ falls steadily as ϕ increases from 0 to 50 deg and beyond.

From Fig. 74, it is seen that the curve for any value of S_i, the maximum shear stress at the point of inflection, falls steadily as ϕ increases from 0 to 50 deg.

Figure 75 is a chart taken from a manual on the design of asphaltic concrete recently published by the Asphalt Institute(*17*). The unshaded portion of this chart indicates the corresponding ranges of values for c and ϕ which asphaltic concrete mixture 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 poor behavior of asphaltic concrete pavements.

It is well known that asphaltic concrete paving mixtures may vary considerably with regard to their stability requirements when employed for different purposes.

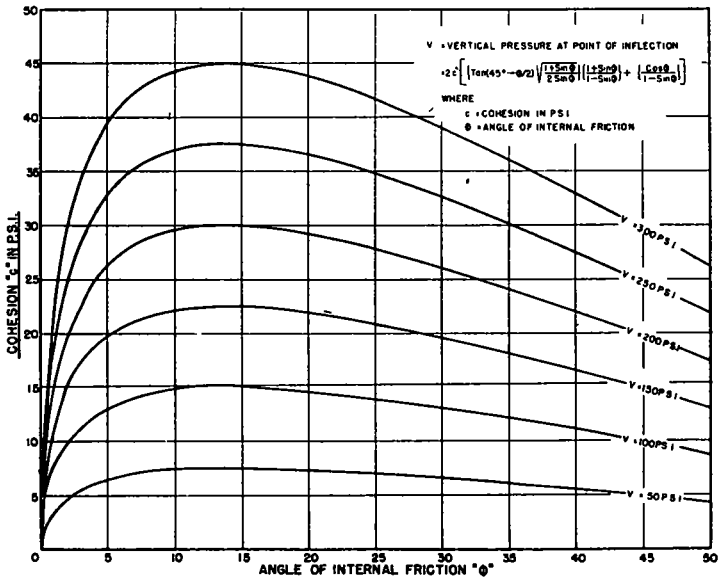

Figure 72. Relationships Between Cohesion, Angle of Internal Friction and Vertical Load at the Point of Inflection for Triaxial Compression Test

Very high stability is needed when the pavement is to be exposed to much starting and stopping of traffic as at stop lights or bus stops. On the other hand, for airport runways, except near the ends, it is generally agreed that more moderate values of stability are acceptable, since factors providing good durability under the relatively limited traffic (as compared with primary highway traffic) require considerable attention. Consequently, a single sharp boundary such as that of Fig. 75 tends to be somewhat illogical, since its location will be such that it favors one of the following conditions:

1. It assures satisfactory performance under severest traffic conditions, and is therefore too conservative for general use.

2. It permits the inclusion of mixtures meeting low stability requirements, and thereby leads to unsatisfactory performance for projects where high stability is required.

3. It will be placed in an average position, where its stability requirements will often be either too severe or too lax.

The diagram of Fig. 75 would be materially improved, therefore if it could be divided into zones of stability, which would assure satisfactory pavement performance over the whole range from moderate to severe conditions of stability in service, since this would lead to greater overall economy in the construction of bituminous paving mixtures designed by the triaxial compression test.

Figure 76 represents a combination of the information of Figs. 73 and 75. The $(V_i - L_i)$ curves of Fig. 73 have been superimposed upon the asphaltic concrete design chart of Fig. 75. It will be observed that a $(V_i - L_i)$ value of 80 psi coincides very well with the lower boundary of Fig. 75. Curves representing lower values of

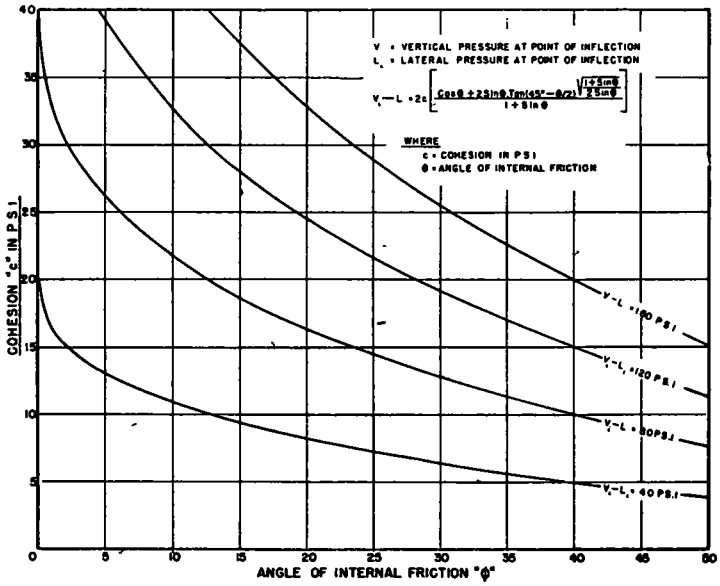

Figure 73. Relationships Between Cohesion, Angle of Internal Friction and $(V_i - L_i)$ for Triaxial Compression Test

$(V_i - L_i)$ than this lie within the portion of the Asphalt Institute chart labelled unsatisfactory. $(V_i - L_i)$ curves for higher values than 80 psi, and to the right of the $\phi = 25°$ ordinate, lie within the area of the chart considered to represent satisfactory design.

Figure 77 combines the information of Figs. 71 and 76, and superimposes graphs for several $(V_i - L_i)$ and L_i values respectively, on the Asphalt Institute diagram of Fig. 75.

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 a study of pavement performance under traffic, coupled with triaxial compression test studies of samples from the pavement.

Figures 76 and 77 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 psi might be adequate where average stability was required. A $(V_i - L_i)$ value of 120 psi 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 $(V_i - L_i)$ value of 40 psi or less might provide adequate stability under traffic. The $(V_i - L_i)$ 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 limitations on this boundary [$(V_i - L_i)$ curve], would exist on the left hand side, and would be marked by the intersection of the $(V_i - L_i)$ curve with the curve for the L_i value corresponding to the

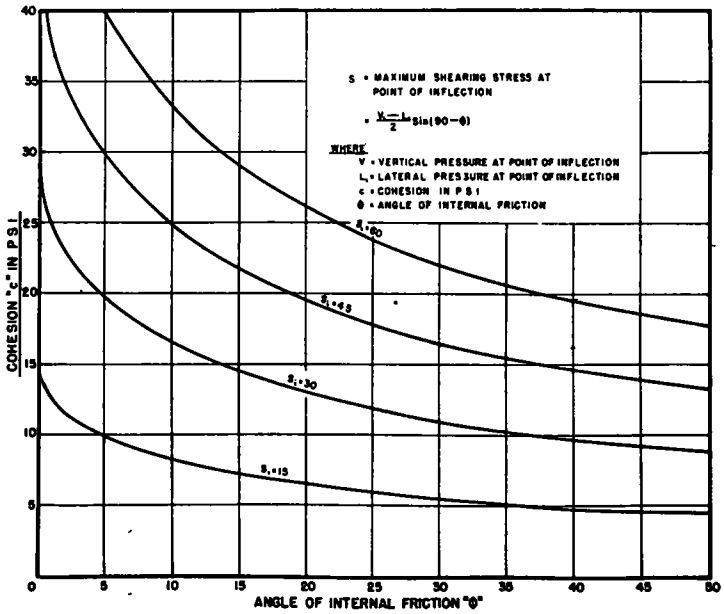

Figure 74. Relationships Between Cohesion, Angle of Internal Friction and S_i for Triaxial Compression Test

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_i value might frequently be to the left of the vertical boundary shown in the Asphalt Institute diagram, Fig. 75. The critical L_i 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 $(V_i - L_i)$ value specified or adopted for the paving mixture.

Bituminous mixtures having combinations of c and Ø (giving higher L_i values) to the left of this critical L_i 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 $(V_i - L_i)$ value specified for stability. The justification or otherwise for sharply defining the critical value of L_i 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 $(V_i - L_i)$ 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 $(V_i - L_i)$ 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 the L_i and $(V_i - L_i)$ curves similar to those of Fig. 77, 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 Ø 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. 77 for designing the stability for bituminous mixtures could also be prepared on the basis of S_1 and L_i values, Fig. 78.

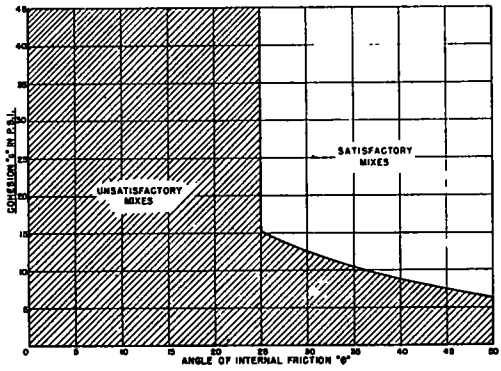

Figure 75. Design Chart for Asphaltic Concrete Based Upon the Triaxial Compression Test (The Asphalt Institute Manual on Hot-Mix Asphaltic Concrete Paving)

SELECTION OF BASE COURSE MATERIALS BY THE TRIAXIAL TEST

It was pointed out earlier, that for similar relative density and moisture conditions, different types of granular bases may have the same supporting capacity per

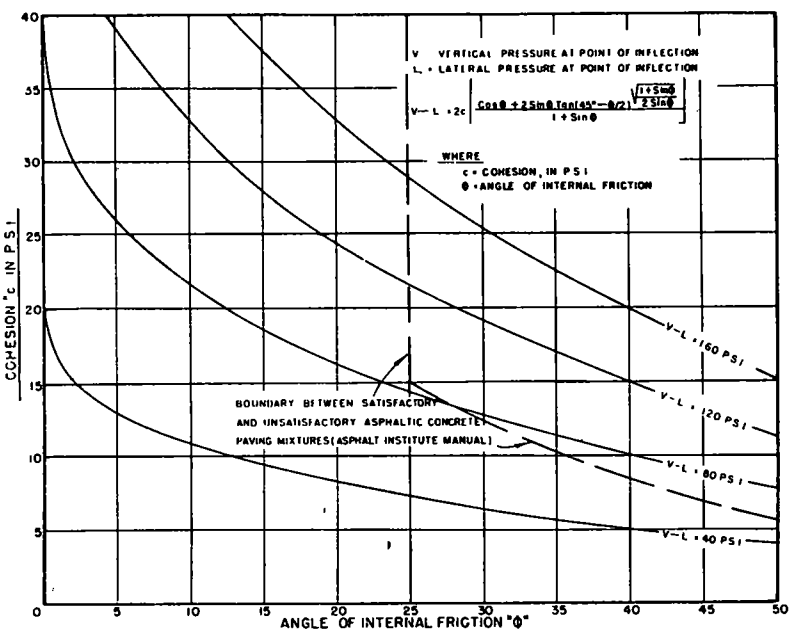

Figure 76. Graph Showing the Boundaries Between Satisfactory and Unsatisfactory Asphaltic Concrete Mixtures Proposed By the Asphalt Institute, and $(V_i - L_1)$ Values

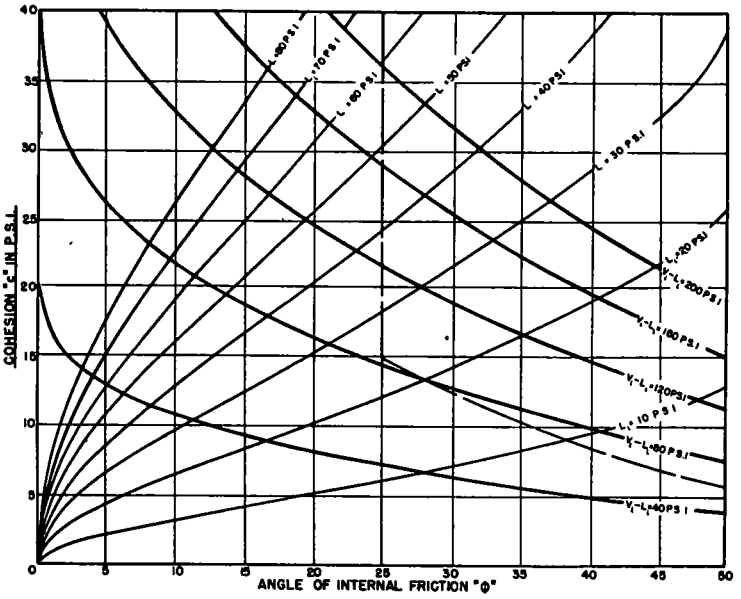

Figure 77. Chart for Asphaltic Concrete Design Based Upon Values of c, Ø, L_i and $(V_1 - L_i)$ Derived From the Triaxial Compression Test

unit of thickness. If this is substantiated by further investigations 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

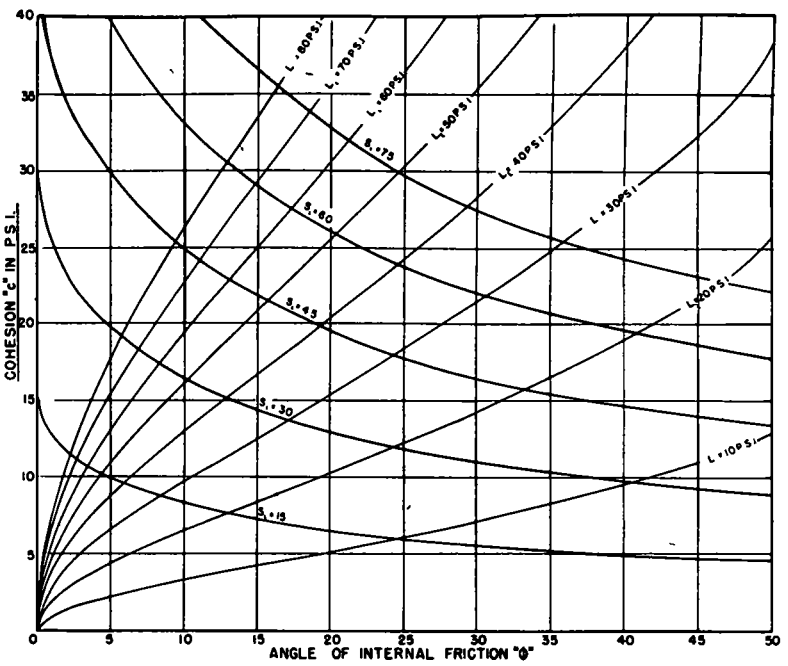

Figure 78. Chart of Relationships Between S_i, L_i, c and Ø From the Triaxial Compression Test

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 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. 79, the shear stress trajectories under a loaded area are indicated. If the shearing resistance along the full line in Fig. 79 were exceeded, base course material under load would move laterally and upward along this trajectory, leading to rutting and probable failure.

The shearing resistance of base course materials can be determined by the triaxial compression test. Those with measurable cohesion can be evaluated by means of Fig. 78, in which the different degrees of stability under load are zoned in terms of S_i and L_i values. For projects or portions of the base course subject to large shear stresses a base course material with a high value of S_i should be specified, and for locations where shear stresses are lower, a material with a lower S_i value could be stipulated. Each S_i curve in Fig. 78 would be bounded toward the left hand side by the L_i 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_i values to the left of this critical L_i 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_i value specified for stability. However, calculations indicate that a considerable decrease in lateral support does not markedly or rapidly lower the S_i stability value of a base course material possessing

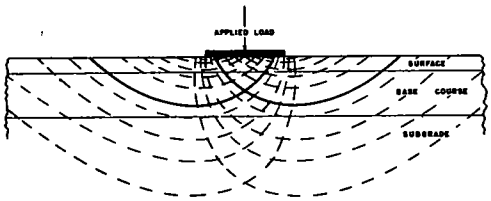

Figure 79. Diagram of Shear Stress Trajectories Under a Loaded Area

measurable cohesion. That is, the stability appears to be much more critical with regard to changes in the S_i 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 measurable cohesion could also be zoned in terms of $(V_i - L_i)$ values, Fig. 77, after the manner described for bituminous mixtures in the previous section.

For base course materials having no measurable cohesion, Fig. 78 could not be employed. The shearing resistance of these is given by the Coulomb equation $s = n \tan \phi$ and

depends on the normal pressure n and the angle of internal friction Ø. 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 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 Ø of the various materials. This angle can be determined by means of the triaxial compression test.

A study is needed, therefore, to determine the S_i and L_i values for base course materials with cohesion, and the values of Ø 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 Ø 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 ranges of loadings and traffic to which highways and airport runways are subjected, than is favored 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.

The stability diagrams of Figs. 77 and 78, which have been developed with regard to base courses and bituminous surfaces, are also applicable to airport and highway subgrades and sub-bases. In addition, these or similar diagrams should find useful application when investigating the stability of elements of earth masses in dams, embankments, foundations, etc., in other engineering fields.

EVALUATION OF LOAD TEST DATA FOR RIGID PAVEMENT DESIGN

It is common procedure at the present time, to base the value of the subgrade modulus k for rigid pavement design on the load which the subgrade will support on a 30-in. diameter bearing plate at a deflection of 0.05 in.

The subgrade modulus is usually determined by means of a simple static load test in which the deflection is observed as the load is increased by measured increments. In actual practice, however, the subgrade under a rigid pavement is subjected to not one load, but to repeated applications of the wheel loads of traffic.

Figure 15 indicates the effect which repetitive loading may have on the value of subgrade modulus k. The k values steadily decrease as the number of repetitions of load are increased.

For heavier wheel loadings, the difference in the above k values for 1 and 100 repetitions would

increase the thickness requirement for a rigid pavement by about 1 in., which may or may not be significant. It is quite probable, however, that the value of k, as ordinarily determined by a simple static load test, should be higher than that shown for 1 repetition of load in Fig. 15.

There is some question as to whether a steel plate 30 in. in diameter is sufficiently large for the determination of the subgrade modulus k. The average slab of rigid pavement has many times the area of a steel plate this size.

Figures 16 and 37 indicate that the unit load supporting value of a soil decreases with increase in size of bearing area for bearing plates up to at least 42 in. in diameter. Consequently, the subgrade modulus k determined with a bearing plate only 30 in. in diameter may be considerably greater than the subgrade support actually provided for a rigid pavement slab. It is to be observed in this connection, that Teller and Sutherland(*11*) suggest that it might be advisable to use bearing plates from 48 to 60 in. in diameter for the determination of subgrade modulus.

It is possible, therefore, that the combination of a simple static load test and the use of a bearing plate (diameter 30 in.) that is much too small, may result in values of subgrade modulus k that are considerably greater than the actual subgrade support that is provided for a rigid pavement.

Figure 42 demonstrates the load deflection curves for subgrade, base course, and flexible wearing surface that are usually obtained at a given test location. In this case the use of a base course has increased the value of subgrade modulus k.

For a number of test locations, the load deflection curves for subgrade, base course, and flexible surface had the shape indicated in Fig. 80. The curves of Fig. 80 are abnormal since the base course gives a smaller value for subgrade modulus k than is provided by the subgrade itself. That is, for these locations, the use of the base course could be a detriment from the point of view of support for a rigid pavement, since the base course would provide less support at 0.05-in. deflection than the underlying subgrade. Whether

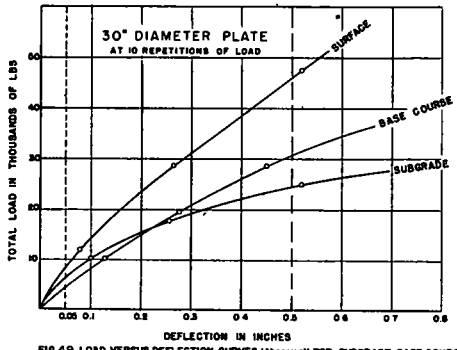

FIG 49 LOAD VERSUS DEFLECTION CURVES (Abnormal) FOR SUBGRADE, BASE COURSE AND SURFACE

Figure 80. Load Versus Deflection Curves (Abnormal) for Subgrade, Base Course and Surface

it was mere coincidence, or whether it represents a commonly occurring phenomenon, it was observed that the condition represented by Fig. 80 occurred most frequently for the macadam type of base course.

From the section immediately below, it follows that the carrying capacity of a given thickness of well compacted base course varies directly with the supporting power of the subgrade upon which it is placed. This principle should also hold true in connection with the improvement of subgrade modulus k by means of properly compacted base courses for rigid pavement design. In this case, the value of increasing the supporting power of the subgrade beneath thoroughly consolidated granular bases for rigid pavements, acquires an economic significance which does not appear to be recognized in rigid pavement design at the present time.

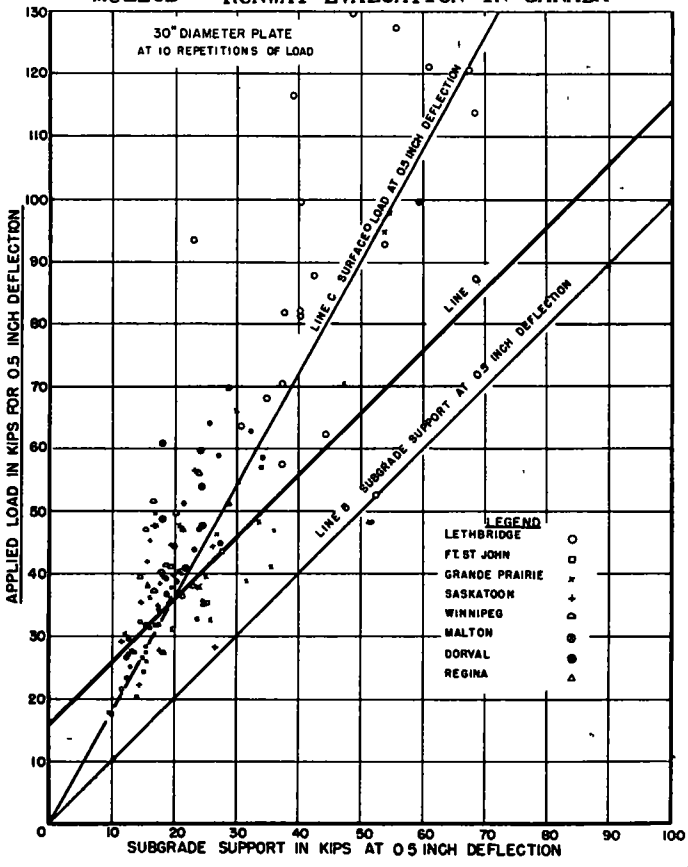

Figure 81. Applied Load Versus Subgrade Support for Overall Thicknesses Corrected to 12 in.

EVALUATION OF LOAD TEST DATA FOR FLEXIBLE PAVEMENT DESIGN

In Fig. 81 the surface load carried by a 30-in. plate at 0.5-in. deflection has been plotted against subgrade support on a 30-in. diameter plate at 0.5-in. 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 in. to about 24 in. at the different test locations, the data of Fig. 81 have been corrected to an overall thickness of 12 in. on a simple proportional basis.

Line B of Fig. 81 has been drawn at an angle of 45 deg and represents on either axis the load carried by the unconfined subgrade on a 30-in. diameter plate at 0.5-in. deflection.

Line C represents the best average line through the points of Fig. 81 and indicates, on the ordinate axis, the load carried by a 30-in. diameter plate on the surface of the runway (corrected thickness 12 in.) at 0.5-in. deflection versus the corresponding subgrade support at 0.5-in. deflection on the abscissa.

Line Q is parallel to Line B.

The following comments are made on the significance of Fig. 81:

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. 81, it would have meant that the base and wearing surface contributed nothing to the load supporting capacity of the structure. That is, the load carrying capacity at 0.5-in. 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 in. 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 in. of a given base and surface increased the overall carrying capacity of a runway by 16,000 lb when the subgrade support at 0.5 in. was 20,000 lb, Line Q indicates that this 12 in. of base and surface would likewise increase the overall carrying capacity by 16,000 lb whether the subgrade support were only 10,000 lb, or 5,000 lb, or 40,000 lb, or any other value. Most of the theories and equations proposed in the past for the required thickness of flexible pavements are implicitely or explicitly based upon this assumption.

Figure 81 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. 81 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. 81 is that *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,* 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 the figure since Line B and Line C both start from the origin and diverge instead of running parallel.

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

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.

5. Figure 81 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. 81, 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.

Campen and Smith(*10*) have re-

ported the effect on overall bearing capacity of a given thickness of base course over two different cohesive soil subgrades. It is significant to note from their data that the base course with the greater supporting power per inch of thickness was the one which had been placed on the stronger subgrade and vice versa.

Hubbard and Field(8) have published the results of some work of theirs, in which the bearing capacities of the subgrade by itself, and after superimposing different thicknesses of asphaltic concrete

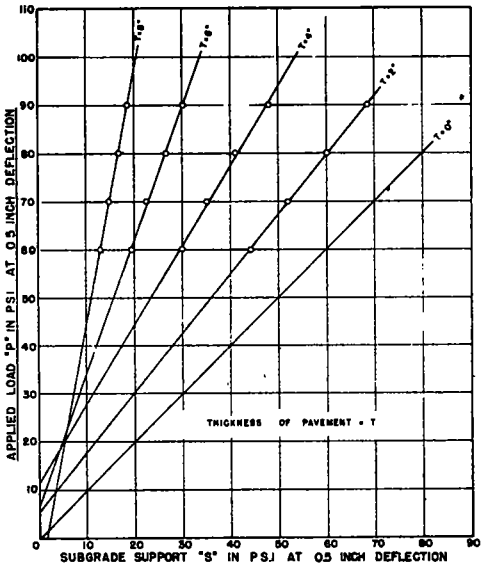

Figure 82. Applied Load Versus Subgrade Support for Asphaltic Concrete (Constructed From Data Contained in the Cover Figure of the Asphalt Institute Research Series No.8)

on the subgrade, were determined for different sizes of bearing plates. Employing the data of the cover figure of Research Series No. 8, and replotting it in a different way, gives Fig. 82. While the lines for different thicknesses do not quite start from the origin or from any other common point, it is to be noted that the graph of surface load versus subgrade support (both at 0.5-in. deflection) is a straight line for each of the various thicknesses of asphaltic concrete and, furthermore, the supporting capacity of each thickness varies directly with the subgrade support, in general.

Klinger developed the following simple equation, which very closely reproduces the Hubbard and Field data of the cover figure of Research Series No. 8:

$$t^2 = K\left(\frac{W}{W_S} - 1\right)$$

where t = required thickness of pavement in inches

W = wheel load applied to pavement at 0.5-in. deflection

W_S = subgrade support at a 0.5-in. deflection for the same bearing area as the surface load

K = a constant having the value of 16.5 for asphaltic concrete

When the data of the cover figure of Research Series No. 8 are plotted in accordance with the Klinger equation, Fig. 83 is obtained. The graph of surface load versus subgrade support is seen to start from the origin for each thickness of asphaltic concrete.

It is to be noted that both Figs. 82 and 83 indicate straight line relationship between surface load and subgrade support. It is to be observed, further, from both figures, that the carrying capacity of any given thickness of pavement varies directly with the strength of the subgrade upon which it is placed. It is strictly true of Fig. 83 and approximately true of Fig. 82 that the load carrying capacity of a given thickness of pavement is doubled when the subgrade support is doubled, and becomes only one-half as great when the strength of the subgrade is halved. Both of these results confirm the conclusions which were indicated by Fig. 81.

INFLUENCE OF BEARING PLATE SIZE ON SURFACE LOAD VERSUS SUBGRADE SUPPORT

In Fig. 84, the influence of the size of bearing plates 30 in. and 12 in. in diameter on the ratio of the load carried by the surface at 0.5-in. deflection versus that supported by the subgrade at 0.5-in. deflection is given for Regina airport. Figure 84 indicates that the best average line through the origin, and through the points for the 30-in. plate, is also quite representative for the location of the points for the 12-in. plate. This means that the relationship between surface load and subgrade support as determined by a 12-in. plate is the same as that which would have been indicated by the 30-in. plate for a weaker subgrade, other conditions remaining the same.

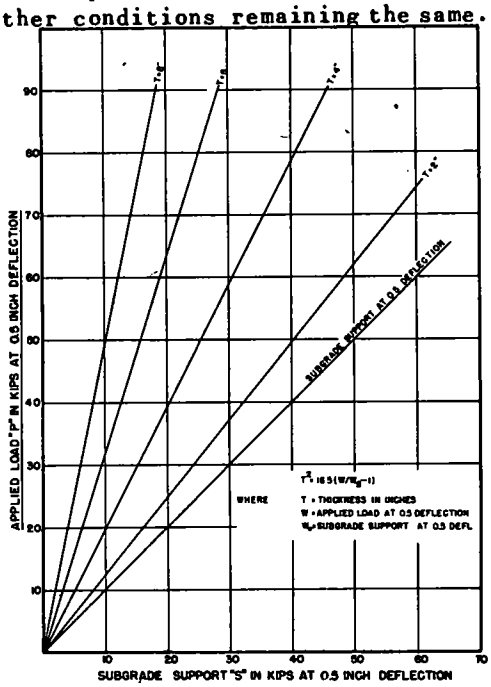

Figure 83, Graph for Klinger Equation Based on Asphalt Institute Data

Data similar to those of Fig. 84 are provided by Fig. 85 for Lethbridge airport for bearing plates 12, 18, 24, 30, 36, and 42 in. in diameter. Again the best averages line through the origin and through the points for the 30-in. plate is,

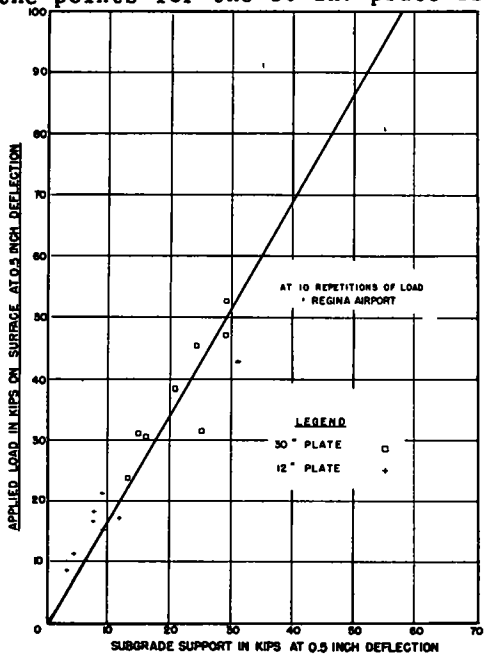

Figure 84. Total Applied Load Versus Subgrade Support for Bearing Plates of Different Sizes (Regina)

generally speaking, a reasonably representative line through the points for the other bearing plate sizes. Consequently, the relationship between surface load and subgrade support determined by bearing plates of different sizes corresponds to that for a 30-in. 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. Expressed somewhat differently, the relationship between surface loads and subgrade support established by a 30-in. diameter plate holds for bearing plates of different diameters all other conditions being the same.

In the technical literature there seems to be very little published information on this parti-

cular problem by other investigators. However, Campen and Smith have been interested in the supporting capacity of base courses per inch of

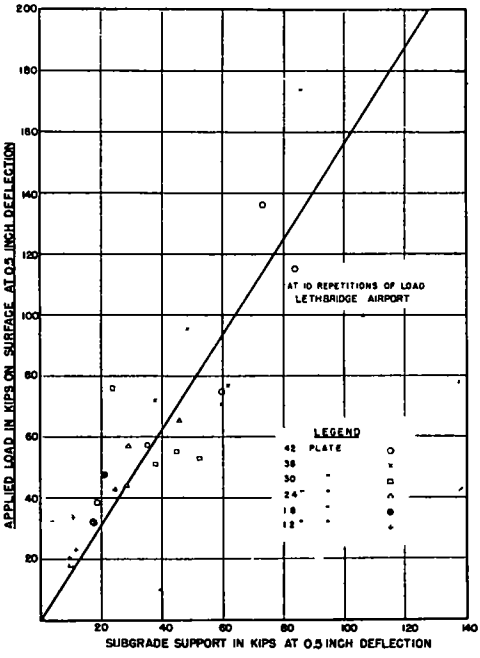

Figure 85. Total Applied Load Versus Subgrade Support for Bearing Plates of Different Sizes (Lethbridge)

thickness and have published the results of several tests they made(9). Figures 86 and 87 have been developed from their data. Figure 86 indicates that the best average line through the origin, and through the point representing surface load versus subgrade support for the 13.5-in. diameter plate, also fits the data for the 9.5-in. and 16.6-in. diameter plates very closely for base course thicknesses of both 6 and 12 in. Figure 87 demonstrates that this is also true for bearing plates 9.5 and 16.6 in. in diameter for another series of tests where the thicknesses of base course were 12 and 18 in.

Consequently, there is reasonable evidence to indicate that the relationship between surface load and subgrade support (both at the same deflection) determined by a bearing plate of one diameter, holds for bearing plates of different diameters (at least over the range of 12 to 42 in. in diameter) if all other conditions remain the same. Establishing this relationship materially simplifies the approach to the development of a method for determining the required thickness of flexible pavements which is outlined below.

A METHOD OF DESIGN FOR THICKNESS OF FLEXIBLE PAVEMENTS

In view of the general scepticism and the verbal brickbats which usually greet the announcement of each newly proposed method of design for determining the thickness of

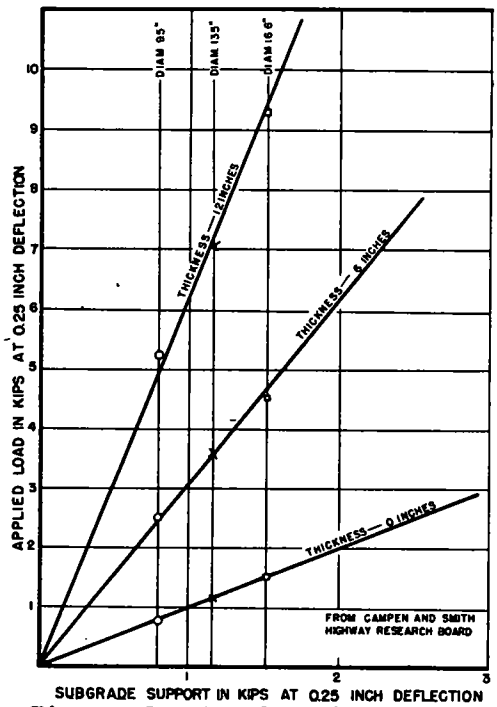

Figure 86. Total Applied Load Versus Subgrade Support for Bearing Plates of 72, 144 and 216 sq in. (Campen and Smith)

flexible pavements, it would probably have been wise to terminate this paper at this point.

However, 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

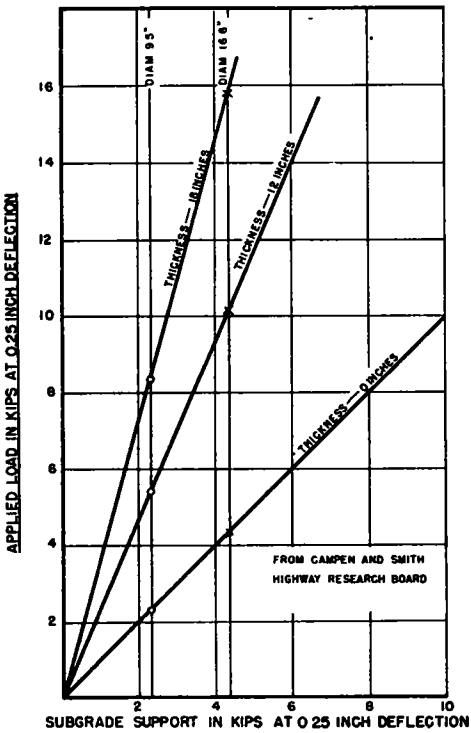

Figure 87. Total Applied Load Versus Subgrade Support for Bearing Plates of 72 and 216 sq in. (Campen and Smith)

to establish the overall thickness of flexible base and surface required to carry airplane wheel loadings of any magnitude. Consequently, this aspect of the investigation cannot be ignored. It also happens that the load test data point to a very definite method for determining the thickness of flexible pavement required to carry any wheel load over subgrades consisting of cohesive soils.

From the load test data 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 loadings 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 airplane wheel loads of any magnitude.

It was demonstrated in Figs. 81, 82, and 83 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.

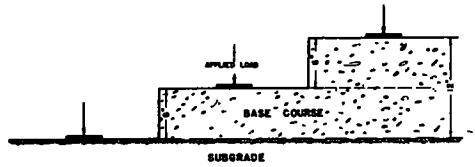

Figure 88. Sketch of Applied Loads on Successive Base Course Layers of Uniform Thickness

Figure 88 can be used to indicate the fundamental principle of this method of design. The load carrying capacity at 0.5-in. deflection of the first 6-in. layer of base course in Fig. 88 is normally greater than that measured on the surface of the subgrade at the same deflection. If the first 6-in. layer of base course is now considered to be the subgrade for the second 6-in. 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-in. layer of base would be greater than that of the second, etc. Figure 89 has been prepared to develop this principle into a simple mathematical equation.

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 linear relationship

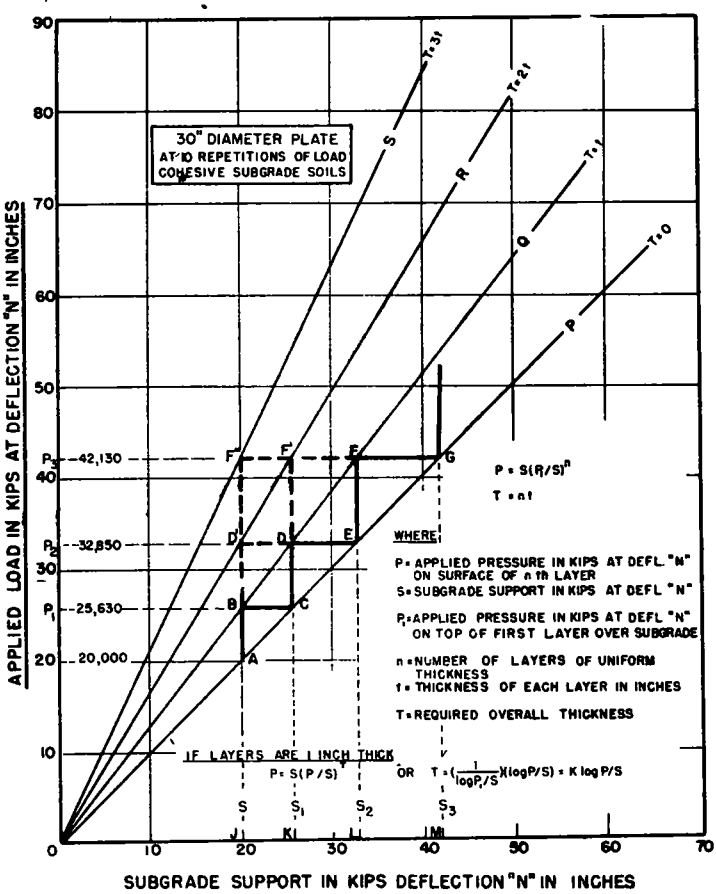

Figure 89. Development of Design Equation for Overall Thickness

The diagram of Fig. 89 has been prepared on the basis of three assumptions, of which the first two are the two conclusions to be drawn from the data of Fig. 81 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

when applied load on the surface of the base is plotted versus subgrade support (Line C of Fig. 81).

The first conclusion above from Fig. 81 implies the assumption that has already been stated with regard to Fig. 88, and which is also required for the preparation of the diagram of Fig. 89, namely:

3. A layer of given base course of specified thickness, normally has a greater load carry-

ing 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, 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 capacity of both subgrade and base course refers 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 afterward superimposed. Soil mechanics is unable as yet to provide reliable information on this latter aspect of the problem.

In the diagram of Fig. 89:

1. Line OP drawn at a slope of 45 deg 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 thickness t, over the subgrade with supporting value S.

4. Line OQ is drawn through Point B. BC is drawn horizontally and BA vertically from B.

5. Since OP has a slope of 45 deg, it follows from geometry that AB is equal to BC, and therefore P_1 is equal to S. Point C therefore, represents the supporting value, $P_1 = S_1$, of a base course of thickness t.
CD is drawn vertically to OQ 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 AB, 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. DE is drawn horizontally through D. From geometry, AD' 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.
EF is drawn vertically to OQ 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 AB, 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_2 which could be carried by a base course of thickness 2t, if the subgrade support were S.

16. Lines OR and OS 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. 89:

$$\frac{BJ}{AJ} = \frac{DK}{CK} = \frac{FL}{EL} - - - \text{etc.} \quad \ldots(1)$$

That is, from what has gone before,

$$\frac{P_1}{S} = \frac{P_2}{S_1} = \frac{P_3}{S_2} = - - \frac{P_n}{S_{n-1}} = \frac{P}{S_{n-1}} \ldots(2)$$

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

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

Therefore, substituting in (2)

$$\frac{P_1}{S} = \frac{P_2}{P_1} = \frac{P_3}{P_2} = - - \frac{P_n}{P_{n-1}} = \frac{P}{P_{n-1}} \quad (4)$$

From which

$$P_2 = \frac{P_1^2}{S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

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

$$\frac{P_3}{P_2} = \frac{P_3}{\frac{P_1^2}{S}} = \frac{P_1}{S} \ldots\ldots\ldots\ldots\ldots (6)$$

or

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

Dividing through by S gives

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

It can be similarly shown that

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

or

$$\frac{P}{S} = \frac{(P_1)^n}{S} \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

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,

$$T = nt \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

where n = number of layers, each of thickness t.

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

$$\frac{P}{S} = \frac{P_1 T}{S} \quad \ldots\ldots\ldots\ldots\ldots (12)$$

or

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

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 has been pointed out, 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 $\log \frac{P_1}{S}$ could be considered a constant, and equation (13) would become,

$$T = K \log \left(\frac{P}{S}\right) \ldots\ldots\ldots (14)$$

where

$$K = \frac{1}{\log \left(\frac{P_1}{S}\right)}$$

A further discussion of the value of the expression $\frac{1}{\log \left(\frac{P_1}{S}\right)}$ from equation (13), is given later.

DETERMINATION OF VALUE FOR CONSTANT K IN DESIGN EQUATION (14)

Before equation (14) can be utilized for design, an average value, or a series of values must be determined for the constant K.

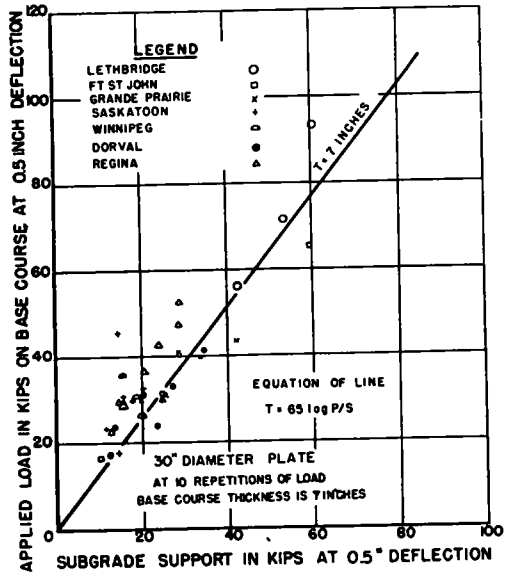

Figure 90. Applied Load in Kips on Base Course at 0.5-in. Deflection Versus Subgrade Support in Kips at 0.5-in. Deflection (T = 7 in.)

Figures 90, 91, and 92 have been prepared for this purpose. The data of these three figures all pertain to 0.5-in. deflection, 30-in. diameter bearing plate, and cohesive subgrade soils.

In Fig. 90, total applied load is plotted against subgrade support for base courses 7 in. thick. The actual thickness in each case varied from about 5 to 9 in., but the data were corrected on the basis of simple proportion to apply to a thickness of 7 in.

In Fig. 91, information similar to that of Fig. 90 is given for base courses of 14 in. in thickness. For this diagram, the actual thickness of base course in the field

varied from 12 to 14 in., but the load test data have been corrected on a proportionate basis to apply to a thickness of exactly 14 in.

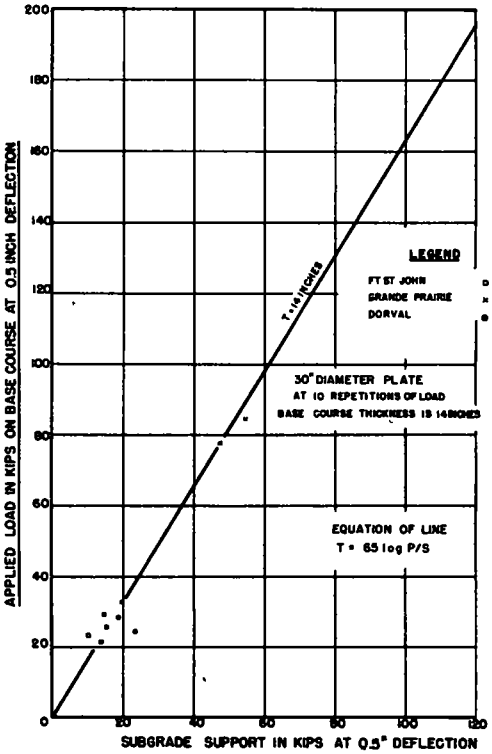

Figure 91. Applied Load in Kips on Base course at 0.5-in. Deflection Versus Subgrade Support in Kips at 0.5-in. Deflection (T = 14 in.)

A base course thickness of 21 in. has rarely been employed for airport runway construction in Canada, and it was necessary to obtain the information for Fig.92 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 in. The thickness of the bituminous wearing surface at these test locations was usually from 3 to 6 in. It was pointed out previously in this paper that 1 in. of bituminous pavement containing liquid asphalt or soft asphalt cement binder, was found to have the same load supporting capacity as about 1½ in. of granular base. Applying this principle to the load test data for 18 in. of pavement and base, gave the load test data in terms of granular base 21 in. in thickness, which appear in Fig. 92.

The straight line relationships through the data of Figs. 90, 91, and 92 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

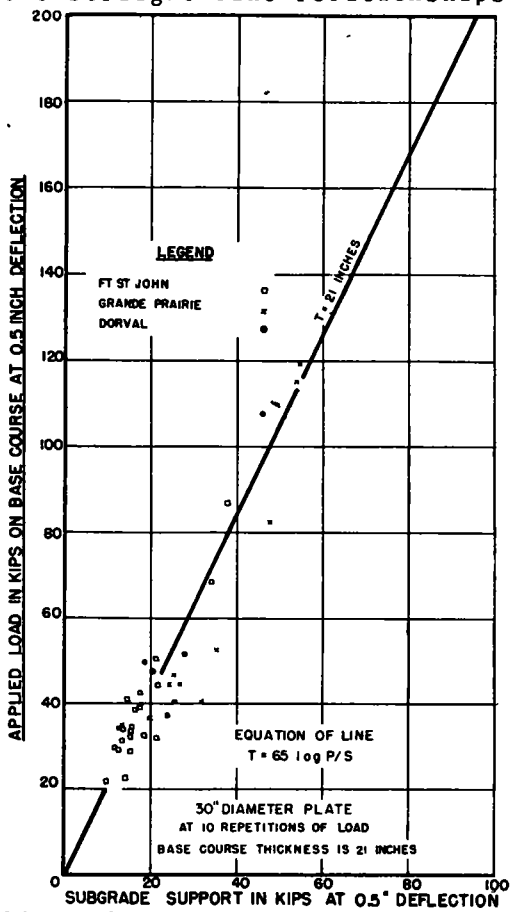

Figure 92. Applied Load in Kips on Base Course at 0.5-in. Deflection Versus Subgrade Support in Kips at 0.5-in. Deflection (T = 21in.)

through the points of these figures represent the design equation

$$T = 65 \log \frac{P}{S} \quad \ldots\ldots\ldots\ldots (15)$$

It is to be observed that equa-

tion (15) represents the particular case of equation (14) in which the value of K = 65.

DESIGN CURVES FOR THICKNESS OF FLEXIBLE PAVEMENTS FOR RUNWAYS

The thickness requirements of equation (14) have been indicated graphically in Fig. 93, to illustrate the influence 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. This is indicated by the continuous line labelled K = 65 in Fig. 93. The influence of three other values of K on the required thickness of granular base, has been shown by means of broken lines.

While Fig. 93 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. Figure 94 consists of a chart of curves showing the required thicknesses of granular base versus subgrade support at 0.5-in. deflection for a wide range of air-

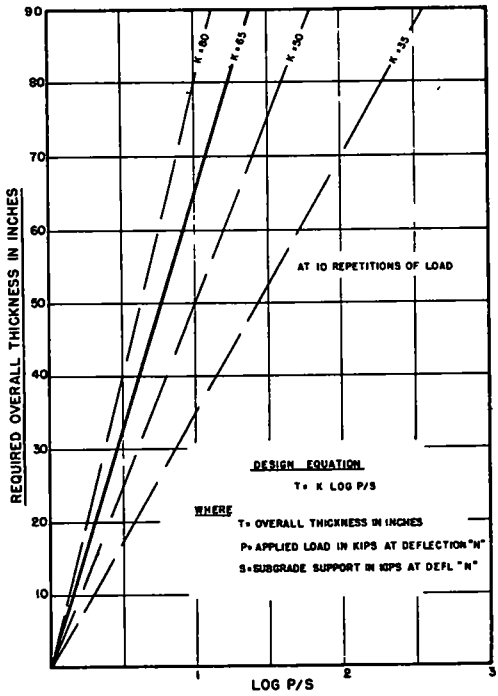

Figure 93. Graph of Overall Thickness Versus Log $\frac{P}{S}$

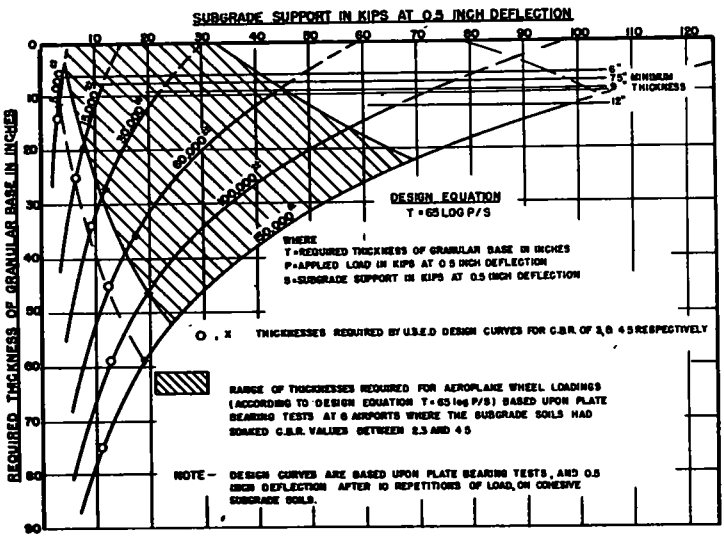

Figure 94. Design Curves for Flexible Pavements for Airplane Wheel Loading (Full Load on Single Tire) - Load Tests

plane wheel loads for airport runways.

The thickness requirements of USED design curves for different airplane wheel loadings over subgrades with soaked CBR ratings of 3 and 4.5, are indicated on the curves of Fig. 94 by means of circles and crosses, respectively. The cross-hatched portion gives the range of thicknesses indicated by design equation (15) for eight Canadian airports at which the soaked CBR ratings of the subgrades varied from 2.2 to 4.6. It should be recalled that the designs represented by the curves of Fig. 94 are based upon 10 repetitions of load, and a subgrade deflection of 0.5 in.

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 USED design. Quantitatively speaking, the combined load test and traffic information obtained during this investigation show 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 USED design, have been functioning satisfactorily in Canada.

The outstanding difference between the chart of Fig. 94 and USED design, however, lies in the fact that for a CBR-3 soil, for example, the USED chart permits only one thickness for any given wheel load. These thicknesses are shown by the small circles on the curves of Fig. 94 for the respective airplane wheel loads. The only departure permitted from the thicknesses represented by these circles can be ascertained from the following two quotations from the Engineering Manual of the Corps of Engineers(1) with regard to the subgrade:

"In general for most materials, the most critical condition ..... will be when the maximum amount of water has been absorbed. For this reason, and in order to secure a conservative design, the design moisture content adopted by the Corps of Engineers is the moisture content attained after the specimens have been immersed in water for a 4-day period while contained in molds and confined with a surcharge equal to the weight of pavement and base that will be above the material.

"In arid or semi-arid regions where the annual rainfall is less than 15 in. and the water table (including perched water table) is at least 15 ft below the surface, the danger of saturation is reduced. The required thickness of pavement and base may be reduced 20 percent for these conditions ..... Complete data substantiating rainfall and water table should be submitted with design analyses."

On the other hand, the chart of Fig. 94 permits the whole range of thicknesses indicated by the design curves to be considered for any individual soil (including those having a soaked CBR rating of 3), depending upon the moisture and density conditions anticipated for it after the pavement has been placed, which may be considerably less than saturation.

It is of interest that the heaviest Canadian design indicated by the shaded portion of Fig. 94 corresponds fairly nearly to USED requirements after the 20 percent reduction in thickness mentioned in the second quotation above has been made in all cases, although for several of the airports included in this investigation, the water table or annual rainfall conditions, or both, were much more severe than those quoted by the USED manual when this reduction was to be considered.

The broken lines on the extreme left and extreme right of Fig. 94 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.

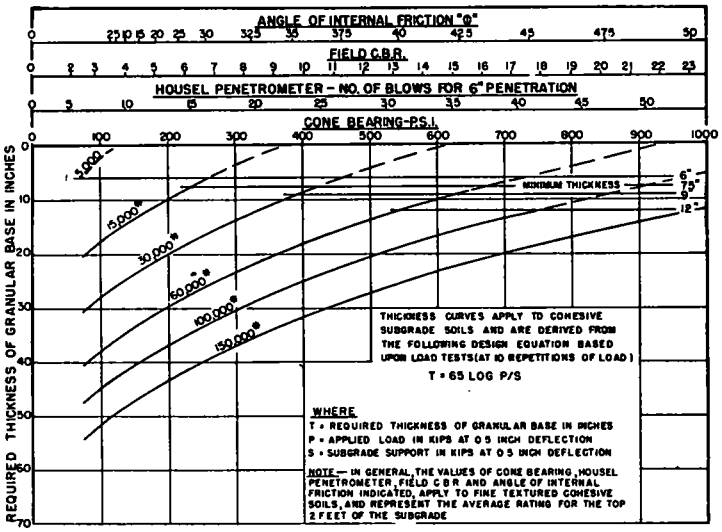

Figure 95. Design Curves for 'rplane Wheel Loadings for Runways for Cohe Subgrade Soils (Full Load on Single Tire) - one Bearing, Etc. Tests

In Fig. 95, the thickness design curves have been drawn on the basis of the four tests, cone bearing, Housel penetrometer, field CBR, and triaxial compression. Figure 95 represents the combined information of Figs. 27, 37, 64, and 94. Figure 95 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.

THICKNESS DESIGN CURVES FOR TAXIWAYS, APRONS, AND TURNAROUNDS

The thickness design curves of Figs. 94 and 95 can be employed to determine the required thickness of base course and surface for any airplane wheel loading for runways, except for the turnaround areas at each end, where the aircraft turn and pause before take-off. They are based upon 0.5-in. 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 runway tests.

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 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. 40 and 41 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. Figures 40 and 41 indicate that the average yield points of the overall runway structures and of the underlying subgrades, occur at 0.225 and 0.26 in., 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 in.

Consequently, if runway design for flexible pavements is to be based upon the subgrade supporting value for 10 repetitions of load at 0.5-in. deflection, 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 at 0.225-in. deflection, the average yield point deflection of the pavement.

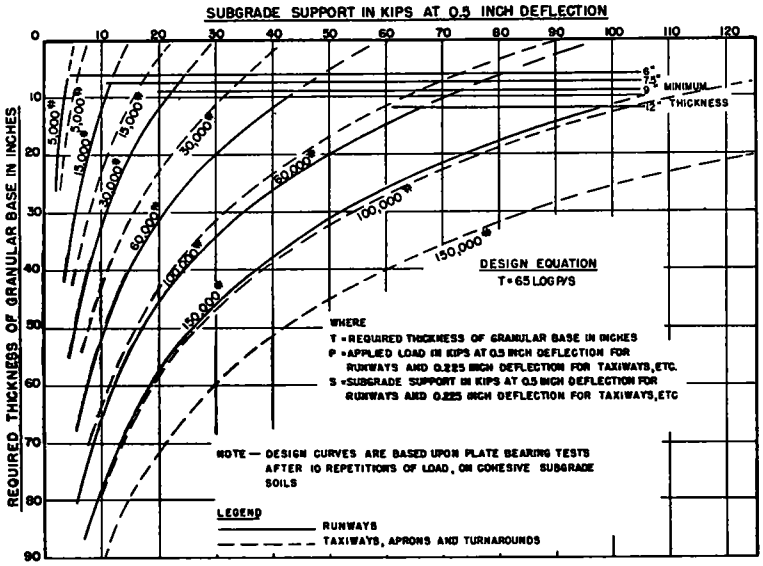

Figure 96. Design Curves for Flexible Pavements for Runways and Taxiways, Etc., for Airplane Wheel Loadings (Full Load on Single Tire) - Load Tests

Figure 96 gives the thickness design curves for a number of airplane 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. 94, and are based upon the design equation $T = 65 \log \frac{P}{S}$ for 0.5-in. deflection. The curves for taxiway,

apron, and turnaround design are those given by this same design equation but based upon 0.225-in. deflection. Both sets of curves can be plotted on the same diagram

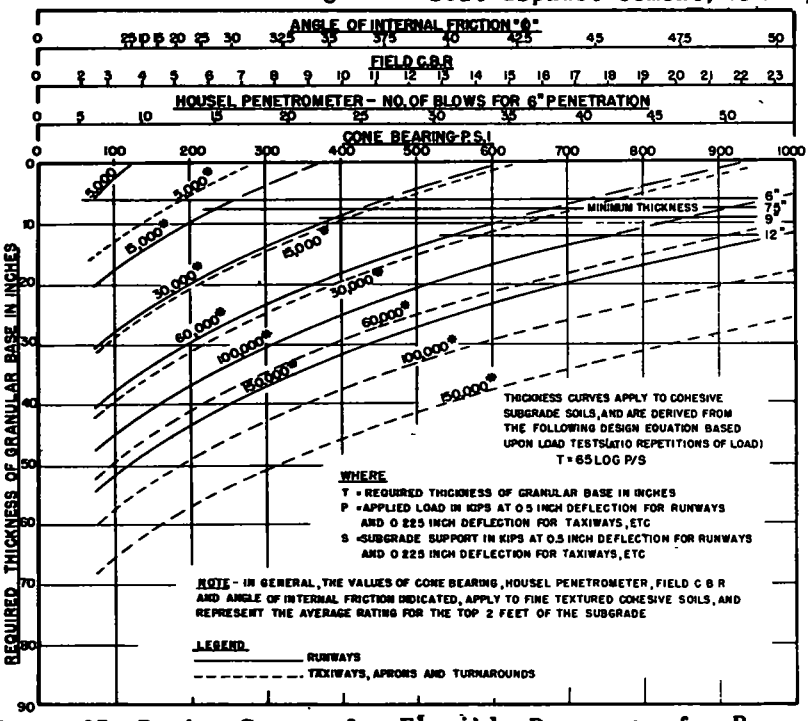

Figure 97. Design Curves for Flexible Pavements for Runways and Taxiways, Etc., for Airplane Wheel Loadings (Full Load on Single Tire) - Cone Bearing, Etc. Tests

(Fig. 96) since there is a constant ratio between subgrade support at 0.5-in. and 0.225-in. deflection for each individual size of bearing plate (Fig. 27).

Figure 97 also gives thickness design curves for runways, and for taxiways, aprons, and turnarounds, but in terms of the four tests, cone bearing, field CBR, Housel penetrometer, and triaxial compression. The curves for runway design are identical with those of Fig. 95. Those for taxiway, apron, and turnaround design are based upon a deflection of 0.225 in., and have been derived by reference to Figs. 27, 37, 64, and 96.

It should be pointed out here that the thicknesses indicated by Figs. 96 and 97 refer to granular base. These may be replaced in the ratio of 1 in. of bituminous surface for 1½ in. of granular base, when the binder is liquid asphalt or soft asphalt cement, or in the ratio of 1 in. of bituminous surface for 2½ in. of granular base in the case of properly designed and constructed asphaltic concrete, penetration macadam, or sheet asphalt. The maximum thickness of bituminous surface to which these ratios can apply, should be taken as 4 in. for the former type, and 6 in. for the latter, until more test data affirms that they may be applied to greater thicknesses. The designer, however, may prefer to specify the thicknesses indicated by Figs. 96 and 97 and utilize the greater supporting value of the bituminous surface as a safety factor.

By making load tests on existing runways, or taxiways, etc., 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. 96, by assuming that the existing surface will be the subgrade for the new base and surface.

The utilization of Figs. 96 and 97 depends upon a knowledge of the available subgrade support. While grading operations should tend in general to provide a uniform degree of subgrade support, measurements of the bearing capacity of the subgrade in place by plate bearing tests, or by cone bearing, Housel penetrometer, field CBR, triaxial compression tests, etc., will provide a series of values at the different test locations on a runway or taxiway, etc., that may vary by several thousand pounds from the low value to the high. If the subgrade were thoroughly evaluated throughout, in conjunction with a pedological soil survey, it would be possible to employ a variable thickness, that is, greater thickness where the subgrade was weak, and less where it was strong. In actual practice this is seldom done, and a uniform thickness is usually specified.

If the lowest value of subgrade support determined is adopted for design, the base and surface will be overdesigned for much of the runway, or taxiway, etc., and the cost of construction will be excessive. If the average value of the subgrade support is chosen, a considerable portion of the runway, or taxiway, etc., may be underdesigned, thereby leading to very high maintenance costs and probable interference with scheduled air traffic. It is suggested, therefore, that the lower 25 percent point (the lower quartile point) be selected as the representative subgrade supporting value for design. That is, the supporting value which is greater than 25 percent, but smaller than 75 percent of the load test results or other measure of subgrade supporting value obtained. This may result in the need for extra maintenance over a small portion of the runway or taxiway, but will avoid the excessive overdesign to which the use of the lowest value of subgrade support might lead.

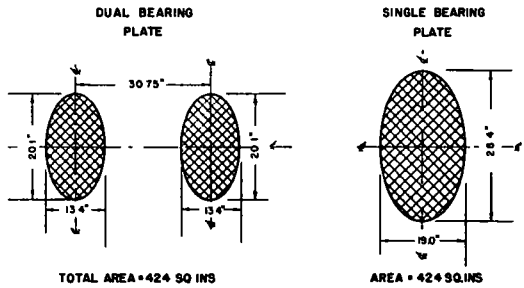

Figure 98. Diagram of Elliptical Steel Plates Employed for Comparison of Single Versus Dual Bearing Plate Tests

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. 98). The spacing between the center lines of the dual plates, 30.75 in., is identical with that for the dual wheels on DC 4 airplanes used by Trans-Canada Airlines. The dual bearing plates were rigidly yoked together so as to function as a unit in a load test.

According to this series of tests, dual wheels of this size and spacing carry from 25 to 35 percent more load than a single wheel with the same contact area at any given deflection over a range of 0.2 to

0.5 in. The difference in load carrying capacity of single and dual wheels appears to decrease as the total thickness of base course and wearing surface is 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 lb for the B-29 superfortress. They report(6) that for thicknesses of base course and surface up to about 10 in., this wheel load of 60,000 lb, if carried on dual tires, had the same effect on runway design as a wheel load of 30,000 lb on a single tire. That is, for base and surface thicknesses up to 10 in., the use of a single tire under the given conditions would increase the design load on the runway by 100 percent 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 percent (maximum) above that for duals.

TABLE 9

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 IN. AFTER 10 REPETITIONS OF LOADING

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

FLEXIBLE PAVEMENT DESIGN FOR HIGHWAYS

When equation (15), $T = 65 \log \frac{P}{S}$ 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. 99 are obtained.

While the small circles and crosses on the four curves were obtained from the USED design chart for airplane wheel loadings for subgrades with soaked CBR ratings of 3 and 4.5 respectively, the basis of the USED 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 CBR ratings of 3 and 4.5 respectively, if not permanently, then at some time during the year. In the California highway survey, thicknesses of base and surface less than those indicated by the circles and crosses, were found to result in failures over subgrades with CBR values of 3 and 4.5 respectively, while greater thicknesses did not.

The shaded portion of the chart represents the range of thicknesses for the four highway loadings, which are indicated by design equation (15), when based upon plate bearing tests at the eight airports where the subgrade soils had soaked CBR ratings between 2.2 and 4.6. 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 CBR of 3, for wheel loads of 12,000, 9,000, and 7,000 lb, but require about 6 in. less for the wheel load of 4,000 lb.

It is of considerable interest that a design equation based upon plate bearing tests at 10 repetitions of load and 0.5-in. deflection should have indicated maximum thicknesses of pavement which conform so nearly to the 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 lb are concerned, therefore, it is obvious that if anything, the thickness requirements indicated by design equation (15), $T = 65 \log \frac{P}{S}$, are somewhat conservative, rather than otherwise.

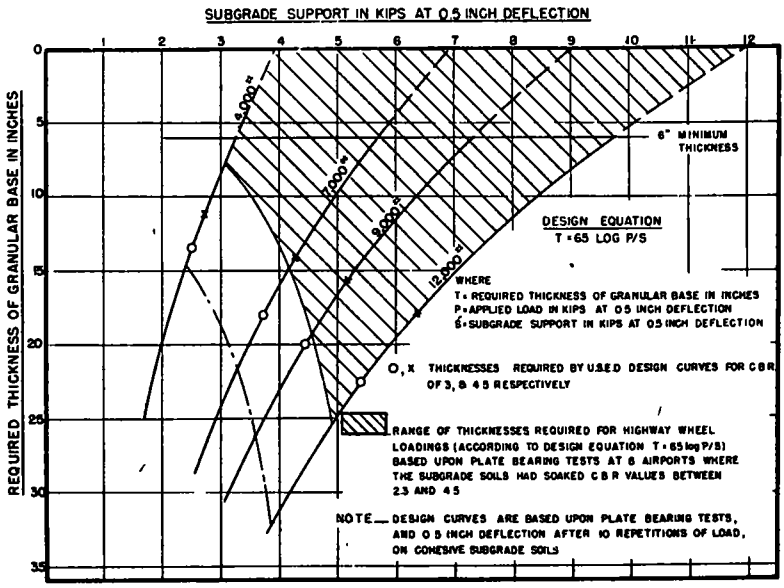

Figure 99. Design Curves for Flexible Pavements for Highway Wheel Loadings (Full Load on Single Tire) - Load Test

It is to be noted that for subgrade soils with soaked CBR values of 3, the four curves of Fig. 99 would contract to the points indicated by the small circle on each curve, if the design recommended by the USED 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 must be considered.

The distance between the circle and the cross along each curve of Fig. 99 represents the range of thicknesses permitted by USED and California designs for subgrades with soaked CBR ratings of 3 to 4.5.

Actual plate bearing tests on Canadian airports with subgrades having soaked CBR ratings between 2.2 and 4.6, have warranted the very much wider range of thicknesses shown by the shaded portion. 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 CBR rating of not over 4.5 the subgrade soil itself had sufficient supporting value in some cases to carry highway wheel loads of even 12,000 lb 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. 99 indicates the thicknesses that would be required for the four highway wheel loadings on the basis of the lowest 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.

Figure 100 provides a chart of thickness design curves for the four highway wheel loadings, based upon cone bearing, Housel penetrometer, field CBR, and triaxial compression tests.

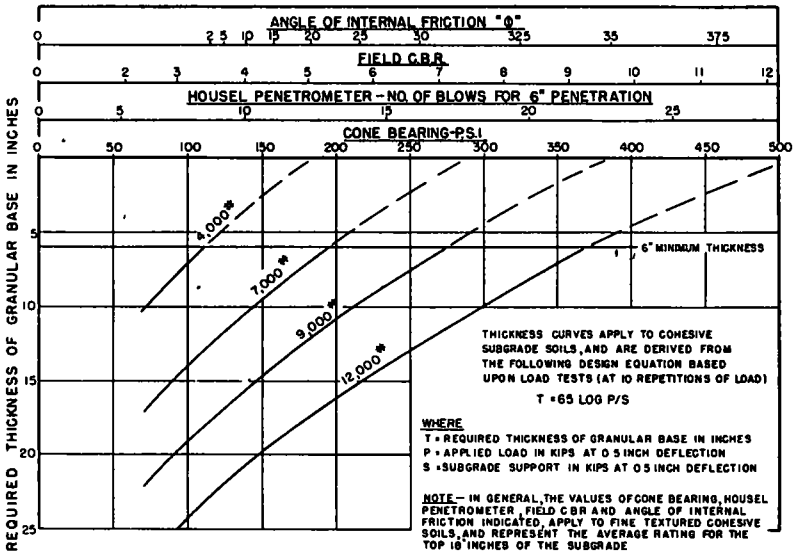

Figure 100. Design Curves for Highway Wheel Loadings on Cohesive Subgrade Soils (Full Load on Single Tire) - Cone Bearing, Etc. Tests

By means of Fig. 100, the required thicknesses of flexible base and surface can be determined in the field by testing subgrades which have been under a pavement sufficiently long for equilibrium moisture and other conditions to have become established, or in the laboratory on the basis of the results of tests on samples of cohesive subgrade soils. By compacting these samples into test moulds in the laboratory at the moisture content and density expected for them in the field (this can be determined from charts similar to those of Figs. 3 to 7 but prepared for each region), and determining their cone bearing, field CBR, Housel penetrometer, or triaxial compression test values

(at least two of these tests should be made as a check, and more than one trial with each test) the required thickness of granular base can be read off the chart of Fig. 100.

When designing the thickness of flexible pavement for a highway project, Figs. 99 and 100 indicate that the value of the subgrade support must be known. If the project is long, the bearing capacity of the subgrade may vary from section to section depending on the nature of the subgrade soil, and upon the topography and drainage. A pedological soil survey will be useful for determining the lengths of the various sections where the subgrade bearing capacity is likely to be reasonably uniform. The subgrade support to be utilized for the design of the thickness of the flexible pavement over each of these sections could be selected on the basis of the lower quartile point, as described at the end of the previous section.

The thickness requirements are again given in terms of granular base. This thickness can be somewhat decreased on the basis of the type of bituminous surface employed in the manner previously described for taxiways, etc.

GENERAL DISCUSSION OF FLEXIBLE PAVEMENT DESIGN EQUATION (14)

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

1. While the thickness design requirements of Figs. 94, 95, 96, 97, 99, and 100 are all based upon a deflection of 0.5-in.(except the curves for the design of taxiways, aprons, and turnarounds of Figs. 96 and 97) it is to be noted that the fundamental development of design equation (14),

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

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

2. The thickness requirements for aprons, taxiways, and turnarounds, given by Figs. 96 and 97 are based upon values of P and S at 0.225-in. 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 in., can be selected for equation (14).

3. The average yield point for the flexible pavements at the 10 airports tested occurred for the load giving 0.225-in. deflection at 10 repetitions of load (Fig. 41). This means that standing or slowly moving wheel loads giving a deflection smaller than 0.225 in. would be supported indefinitely, while those causing a deflection greater than 0.225 in. would cause continuous settlement and eventual deep rutting and failure. This explanation follows from Professor Housel's studies in connection with the design of foundations for buildings and similar engineering structures (*19*). Figure 41, therefore, indicates that a load giving a deflect-

ion of 0.225 in. is the largest that could be supported by taxiways, aprons, or turnarounds where aircraft are stationary or moving slowly.

For the rapidly moving wheel loads of aircraft on runways, experience at Canadian airports indicates that an adequate design may be based upon the loads giving a deflection of 0.5 in. at 10 repetitions of load in a plate bearing test. This does not imply, however, that a deflection of 0.5 in. actually occurs under these rapidly moving wheel loads. It probably means rather, that a moving wheel load great enough to give a deflection of 0.5 in. at 10 repetitions of load when standing still causes a deflection not greater than 0.225 in. (the yield point at 10 repetitions of load) when moving rapidly during take-off or landing.

It is to be noted from Fig. 27 that the stationary load supported at 0.5-in. deflection is approximately 50 to 60 percent greater than that supported at 0.225-in. deflection. Consequently, the use of 0.5-in. deflection (at 10 repetitions of load) for runway or highway design, does not necessarily mean that this deflection actually occurs under rapidly moving wheel loads. It probably implies, rather, that the ratio between the magnitude of rapidly moving wheel loads versus the magnitude of stationary wheel loads at 0.225-in. deflection can be approximately 1.5 to 1.6 without exceeding the yield point deflection of 0.225 in. in either case.

4. There will probably be some criticism of the use of 0.225-in. deflection as representing the yield point deflection for all flexible pavements, since the true yield point deflection probably varies from project to project, depending upon the nature of the subgrade soil, and of the base and wearing course material, etc. This is also true of the use of 0.26-in. deflection as the average yield point deflection for subgrades. The validity of such criticism is recognized.

On the other hand, unless we are willing to accept average values, which have been carefully obtained from the study of data from a large number of projects, for the many variables that enter into flexible pavement design for airport and highway construction, overall design charts such as those of Figs. 96, 97, 99, and 100 would have very limited application.

The alternative would be to design each individual project on the basis of tests made at or for the site. Probably this is what should be done, but it would at the same time seldom be practical because of the cost of the amount of testing that would be involved. Such testing would have to be quite comprehensive, for there is good reason to believe that yield point and other data derived from a limited number of tests could lead to considerably greater error than a design based upon the charts of Figs. 96, 97, 99, and 100, because the vagaries of data that are frequently introduced by too few tests, which may lead to serious error, are ironed out when average trends are developed from a large mass of data for many projects.

It has been our experience, for example, that the yield point values determined by Professor Housel's method may vary over a wide range for different locations at a single airport, even when the subgrade soil type is constant throughout as shown by the soil survey. These variations are probably due to the idiosyncrasies normally encountered when testing such a difficult material as soil. Unless exhaustive load tests with

different plate sizes are to be made on each runway or airport, the investigator is faced with the necessity for choosing a representative yield point from the information he has obtained. If his load test data are limited, the yield point deflection selected in this manner may be much less accurate than that determined from the reasonably large mass of data which led to the development of Figs. 40 & 41.

The two alternatives for flexible pavement design therefore appear to be:

(a) The use of average design charts which have been prepared from studies of data made for a large number of projects, but which may therefore not always be strictly applicable in each individual case.

(b) Abandonment of average design charts and the substitution of an individual design procedure for each project, based upon a number of tests made on or in connection with the site, to determine the influence of each variable to be considered in design. This introduces the possibility of inaccuracy because of the inconsistencies that are frequently observed in the data from a limited number of tests. These may lead to a greater overall error in design than would result from the use of design charts based upon average data from a large number of projects.

The nature and number of tests that would be required for the design of each individual project without reference to the information obtained at others, are so costly and time consuming, that any organization attempting this procedure would in a very short time probably find itself seeking average values upon which overall design charts similar to those of Figs. 96, 97, 99, and 100 could be based.

An outstanding advantage of the charts (particularly Figs. 97 and 100) or of charts similar to these, lies in the fact that the overall design for a flexible base and surface for any project, can be based upon very simple tests which can be rapidly performed in the field or in the laboratory, as circumstances require.

5.

(a) Examination of design equation (14)

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

indicates that the thickness of flexible base and surface required to carry any given wheel load P over any subgrade support S, might be reduced by one or the other or both of the following procedures: (1) increase the value of the subgrade support S; (2) lower the value of constant K.

(b) Apart from the provision for adequate drainage, the only simple method available to engineers at the present time for increasing the subgrade support S is subgrade compaction. The question that very naturally follows is, how much improvement in subgrade support might be reasonably expected as a result of subgrade compaction?

The possibilities in this direction are illustrated by the graphs of Figs. 101 and 102, in which the curve of cone bearing values has been plotted versus the modified AASHO compaction curve for two soils. The cone bearing values were obtained on the compacted samples employed for determining the compaction curve in each case.

Figure 101 indicates that at optimum moisture and 100 percent modified AASHO maximum density (peak of the compaction curve), the sample of heavy clay soil to which it pertains has a cone bearing value of 1400 psi. The

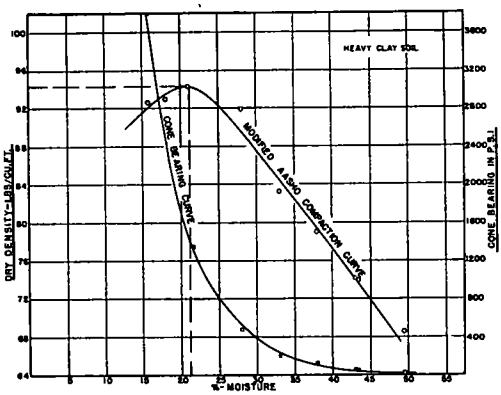

Figure 101. Influence of Soil Moisture and Density on Cone Bearing Values (Heavy Clay)

cone bearing value measured in the field for the subgrade under the pavement in the vicinity of the sample to which the data of Fig. 101 refer, was about 210 psi.

(c) The thickness design curve for runways labelled "60,000 lb" in Fig. 97 shows that for a cohesive soil subgrade with a cone bearing value of 210 psi, a thickness of 29 in. of granular base is required. With a cone bearing value of 1400 psi however, Fig. 97 indicates that the subgrade itself would have sufficient supporting value to carry an airplane wheel load of 60,000 lb without any base or surface. Even if the subgrade could be improved by compaction to have a cone bearing value of 640 psi, only the minimum thickness of 9 in. of base course would be required. This represents a possible reduction of base course thickness of 20 in. in this case, as a result of subgrade compaction.

(d) Figure 102 gives a cone bearing value of about 3700 psi corresponding to the moisture and density conditions for the peak of the compaction curve. No field cone bearing values are available for comparison. However, from Fig. 7, a field moisture of about 50 percent greater than the optimum, or about 14 percent, could be expected for cohesive subgrades in the area from which it came. Figure 102 demonstrates that at a moisture content of 14 percent, a cone bearing value of only about 200 psi could be expected

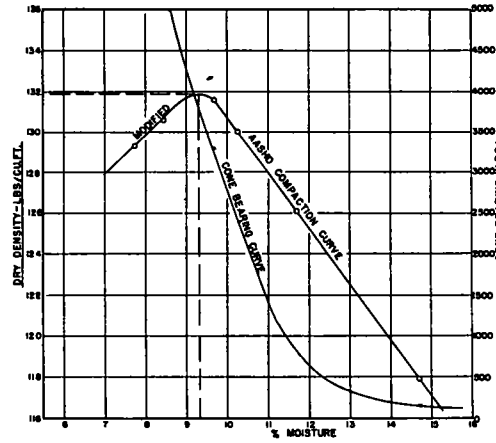

Figure 102. Influence of Soil Moisture and Density on Cone Bearing Values (Sandy Clay Loam Soil)

for this soil. Figure 97 shows that a subgrade of this soil, if compacted within the vicinity of maximum density, could support wheel load of 60,000 lb or greater without any base or pavement.

(e) In Fig. 103, the curve of Ø values from the triaxial compression test versus the modified AASHO compaction curve for a heavy clay soil is shown. By reference to Figs. 97 and 103, the influence on subgrade support of increased density on the wet side of the compaction curve is seen to indicate marked economy

in base course thickness requirements.

(f) The improvement in subgrade bearing capacity, which Figs. 101, 102, and 103 indicate to be possible through attainment of higher subgrade density on the wet side of the compaction curve,

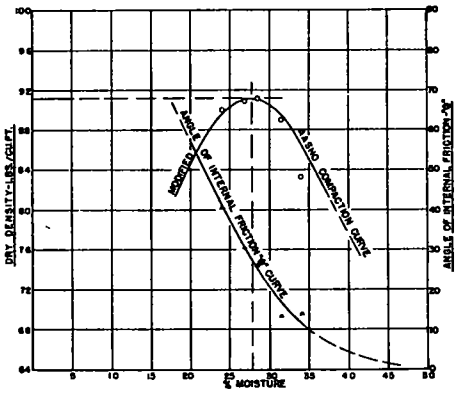

Figure 103. Influence of Soil Moisture and Density on Angle of Internal Friction "Ø" Values

seems to be of much more importance to highway and airport engineers than concern for higher density on the dry side of the compaction curve, since Figs. 3 and 7 indicate that in the majority of cases where the field density is less than 100 percent modified AASHO maximum, the field moisture content of cohesive subgrade soils will be greater than 100 percent of the corresponding optimum moisture.

(g) The cone bearing test is influenced by the nature of the soil within only an inch or so of the cone. The plate bearing test on the other hand, integrates and registers the influence of all subgrade layers over the full depth affected by the applied load.

The problem, therefore, is to determine by means of plate bearing tests, the depth of subgrade which must be compacted to 100 percent to 95 percent, or to a similar percentage of modified AASHO density, to obtain the marked improvement in subgrade support through compaction which is indicated to be possible by the cone bearing or triaxial compression tests (Figs. 101, 102, and 103).

It may be of course that investigation will show that the apparent improvement of subgrade bearing capacity by means of greater compaction, which is indicated to be possible by the cone bearing and triaxial compression test data of Figs. 101, 102, and 103, has been somewhat exaggerated because of certain limitations of these tests for soils that have been highly compacted in a mold in the laboratory. For example, somewhat different relationships may exist between plate bearing tests on a 30-in. diameter plate at 0.2-in. deflection versus cone bearing or triaxial compression tests for soils which have been highly compacted in a laboratory, than those established in Figs. 47 and 60, respectively. This too should be checked by additional study since one or more of the cone bearing, Housel penetrometer, field CBR, or triaxial compression tests, when made on laboratory compacted samples, may be more indicative of the possible improvement of subgrade bearing capacity by compaction in the field, than others of these four tests.

There is a corollary problem in that these layers of highly compacted subgrade soils require study over a period of years to learn whether or not they remain in their consolidated condition after compaction, and if not, what measures would be required to maintain them in this condi-

tion. No systematic large scale study appears to have been devoted to this question, although some controversial isolated data have been reported.

There is a further problem of great practical importance. This consists of developing an economical method for drying soil in the field in those areas of frequent and heavy rainfall where cohesive soils have moisture contents considerably above the optimum in order to take advantage of the greatly increased subgrade bearing capacity that apparently results from compaction at optimum moisture to 100 percent modified AASHO density.

(h) The information of Figs. 97, 100, 101, 102, and 103 might be taken to indicate that since the subgrade itself, if adequately compacted, may have sufficient bearing capacity to carry airplane or highway wheel loadings, much of the cost of construction of base course may be the price which airport and highway engineers are paying for not requiring a high degree of subgrade compaction.

Probably, however, it would be more correct to point out that by means of the information in these or similar graphs, the engineer can work out the combination of subgrade compaction and base course thickness which will carry the anticipated traffic at the lowest overall cost of construction and maintenance for each runway or highway.

(i) The value of the constant K in equation (14) appears to depend entirely on the nature of the base course material. On the basis of all load test data obtained so far, K seems to have an average value of 65 for the base course materials and construction procedures employed for the airport runways studied.

According to the data obtained in this investigation, the value of K may be largely independent of the composition of the base course material, but there is some evidence that it might be substantially influenced by the degree of compaction of the base course.

The average densities of the base courses at several of the airports tested, together with the general composition of the base course materials, are given in Table 10.

It will be observed that the average field densities (in place) of the base courses at the airports listed in Table 10 are below 100 percent modified AASHO with the exception of that at Regina, which is 103 percent.

From the best average line through the data of Fig. 84 it can be calculated that the base course for Regina Airport has a K value of about 35. This is considerably smaller than the average K value of 65 for the base courses for all airports tested so far (Figs. 90, 91, and 92).

By referring to design equation (14), it is apparent that a lowering of the value of K from 65 to 35, would reduce the thickness of base required to carry any given wheel load by very nearly one-half.

It would be very much worthwhile, therefore, to determine the characteristics of the base course which influence the value of K, and to learn what modifications of present procedures of base course construction are required in order that a lower value of K

TABLE 10

COMPOSITION AND DENSITY OF BASE COURSES AT SEVERAL AIRPORTS

Airport	Composition of Base Course Material	Average Field Density in place (dry) lb per cu ft	Modified AASHO Density (dry) lb per cu ft	Field Density as Percent of Modified AASHO Density
Fort Nelson	pit run gravel	140.7	145.7	96.7
Fort St John	crusher run gravel	139.8	152.7	91.7
Grande Prairie	mechanical stabilization	133.4	144.9	92.3
Lethbridge	crusher run gravel	129.3	143.9	90.0
Saskatoon	crusher run gravel	133.2	143.2	93.1
Regina	mechanical stabilization	151.8	147.8	102.7
Winnipeg	mechanical stabilization	145.3	148.4	97.8
Dorval	waterbound macadam	not determined		

in design equation (14) could be justified.

Such an investigation would require a study, by means of bearing plate tests, of the influence of the nature of the base course material, its composition, its moisture content, and its density, on the value of K.

A GENERAL EQUATION FOR FLEXIBLE PAVEMENT DESIGN

1. Equation (14), $T = K \log \frac{P}{S}$, was developed upon the basis that a layer of given base course material 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. 89).

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 ft thick would probably add very little to the supporting value of a gravel deposit 100 ft deep over clay, assuming that layer and deposit were alike in every respect. Consequently, the graph of Fig. 93, instead of being a straight line, would probably be a curve which is generally concave upwards as demonstrated by the broken line curve of Fig. 104. 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. 93, but should vary with the depth of base course required.

It is necessary, therefore, to reconsider equation (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:

$$\overset{\text{I}}{[\ K\ ]}\ \overset{\text{II}}{[\ \log \frac{P}{S}\ ]}$$

Part II, $\log \frac{P}{S}$, seems to be independent of the depth of base course and is therefore valid as it stands. Experimentally, it follows directly from the straight line relationship between surface load versus subgrade support for different thicknesses of base previously shown. It also follows from Fig. 89, and from the development of equations (4) to (14), that the expression $\log \frac{P}{S}$ is independent of the thickness of base course. For example, the geometrical arrangement of steps in Fig. 89 for a base course layer of thickness t, from which the expression $\log \frac{P}{S}$ was derived, holds for a base course of any given depth from a fraction to a multiple of t, since the relationship between surface load and subgrade support for a base course of any given thickness is expressed by a straight line through the origin, e.g. Figs. 90, 91, and 92.

The expression $\log \frac{P}{S}$ of equation (14) therefore results from the straight line relationship obtained 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. Item 5(i) of the preceeding section however, indicated that the value of K for Regina Airport 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 $K = \frac{1}{\log \left(\frac{P_1}{S}\right)}$. The right hand expression 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 on the composition, moisture content, and density of the base course material.

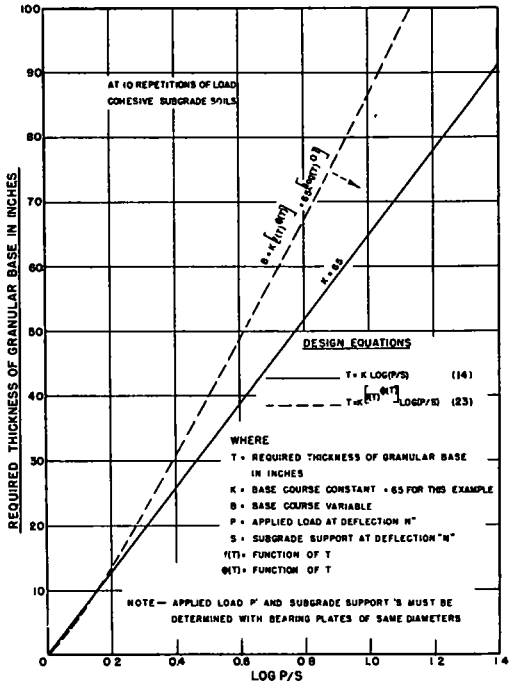

Figure 104. Influence of the Nature of the Base Course Variable on Required Thickness for Flexible Pavement Design

It has been pointed out that the value of the right hand side of equation (14) must vary with the depth of base course. It has already been indicated that Part II of the

expression on the right, i.e. $\log \frac{P}{S}$ appears to be independent of the depth of 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 more general case, the required thickness of base course would be given by the following expression:

$$T = \left(\frac{1}{\log \frac{P_1}{S}} \right)^{f(T)} \log \frac{P}{S} \quad \ldots(16)$$

where T = required thickness of granular base in inches.

P_1 = load supported at any 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

$$T = K^{f(T)} \log \frac{P}{S} \quad \ldots\ldots\ldots\ldots(17)$$

in which

$$K = \frac{1}{\log \frac{P_1}{S}} \quad \ldots\ldots\ldots\ldots(18)$$

where K = base course constant, and for any given base course it has the value given by the expression $\frac{1}{\log \frac{P_1}{S}}$. 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 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:

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

in which

$$f(T) = (\log T)^r \quad \ldots\ldots\ldots(20)$$

That is, f(T), the function of the thickness T required, is the logarithm of the thickness T raised to the rth power, where r is a fraction having a value between zero and one.

There is a possibility that the exponential term r of the 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

$$r = \phi T \quad \ldots\ldots\ldots\ldots(21)$$

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

$$T = K^{[f(T)\phi T]} \log \frac{P}{S} \quad \ldots\ldots(22)$$

or

TABLE 11

INFLUENCE OF THE MAGNITUDE OF $\phi(T)$ ON THE VALUES OF $B = K^{[(\log T)^{\phi(T)}]}$ FOR THE THICKNESSES INDICATED, WHEN THE VALUE OF K IS TAKEN AS 65 FOR THE PURPOSE OF ILLUSTRATION

Values of $B = K^{[(\log T)^{\phi(T)}]}$ for the Thicknesses Indicated

Thickness T in.	$\phi(T)$ 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	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0
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

$$T = K^{[(\log T)^{\phi(T)}]} \log \frac{P}{S} \quad \ldots(23)$$

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

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

$$T = B \log \frac{P}{S} \quad \ldots\ldots\ldots\ldots(24)$$

in which

$$B = K^{[(\log T)^{\phi(T)}]} \quad \ldots\ldots(25)$$

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 $B = K^{[(\log T)^{\phi(T)}]}$ is shown in Table 11 for a wide range of values of $\phi(T)$, when the value of K is taken as 65 by way of example.

From the data of Table 11, it is apparent that values of $B = K^{[(\log T)^{\phi T}]}$ of equations (23) or (25) depart very little from the value of K in equation (14), for small values of $\phi(T)$. For values of $\phi(T)$ greater than about 0.1 on the other hand, the value of $B = K^{[(\log T)^{\phi(T)}]}$ may deviate widely from the corresponding value of K.

For each vertical column of data in Table 11, it will be observed that $\phi(T)$ has a constant value. If

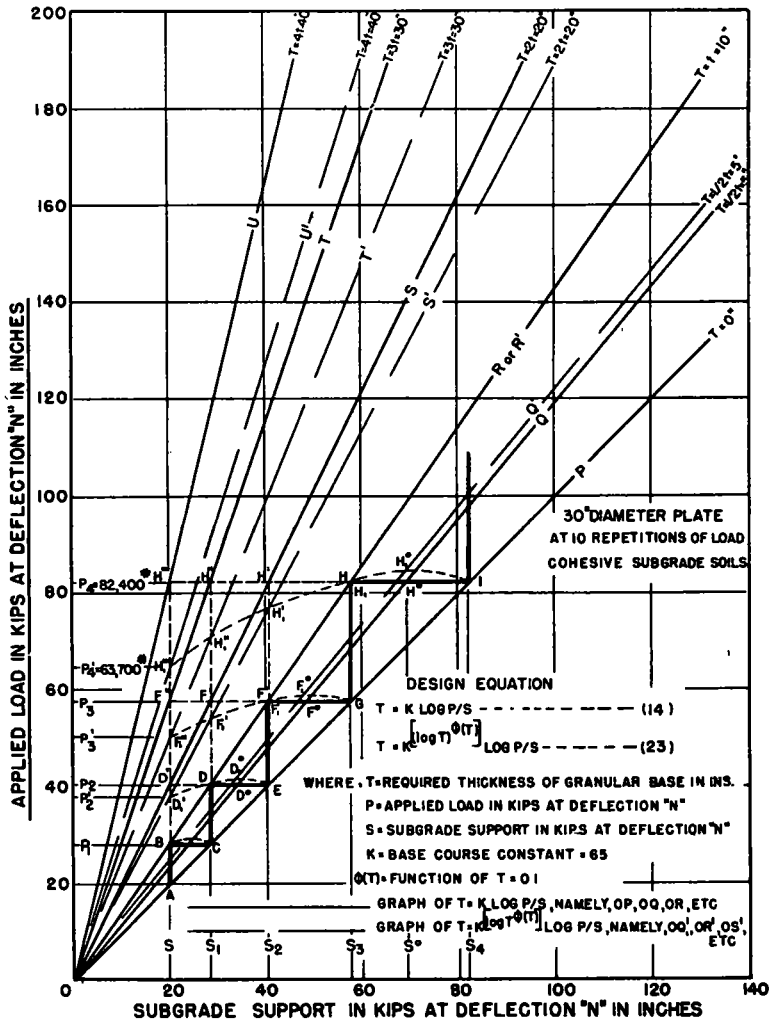

Figure 105. Influence of the Nature of Flexible Pavement Design Equations on Applied Load Versus Subgrade Support

$\phi(T)$ should vary with the thickness T as its symbol permits, the influence of thickness T on the value of $\phi(T)$ can be observed by reading the data of Table 11 from top to bottom in a generally diagonal direction.

It should be noted particularly that when $\phi(T) = 0$, 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. 104. 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 $\phi(T) = 0.1$. 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. 104 would be exaggerated, however, if the value of $\phi(T) = 0.1$. employed by way of example is considerably higher than experimental data would indicate.

Figure 105 is similar to Fig. 89 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 40 in. of base course, respectively, are drawn on the basis of equation (14) 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 in. respectively, pertain to equation (23) using a value of $K = 65$, and a value of $\phi(T) = 0.1$.

For any given value of thickness greater than 10 in., it is obvious from Fig. 105 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 subgrade support at a higher value of P, than so the broken lines of OS', OT', and OU', respectively. For a subgrade support of 20,000 lb for example, and for a thickness of 40 in., P_4 given by equation (14) is 82,400 lb, whereas P'_4 given by equation (23) is 63,700 lb.

The horizontal straight line I, H^o, H, H', H", H"', of Fig. 105, shows the constant value of the applied load P_4, which would be supported by various thicknesses of base course 0, $\frac{1}{2}t$, 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_1o, H_1, H_1', H_1'', and H_1''', on the other hand, demonstrates the variable nature of the applied load which is supported by base coarse thicknesses of 0, $\frac{1}{2}t$, t, 2t, 3t, and 4t, over the same degrees of subgrade support, S_4, S^o, S_3, 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", and the curved line G, F_1^o, F_1, F_1', F_1'', and by E, D^o, D, D', versus E, D_1o, 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. 105, since future experimental data might indicate that a value of $\phi(T) = 0.1$ is too high to use in equation (23).

A careful and comprehensive research program would be required to evaluate the expression

$$B = K^{[(\log T)^{\phi(T)}]}$$

of equation (25), 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 $\phi(T)$ have been established, the thickness requirement T given by equations (23) or (24) can be determined very easily by a series of successive approximations, as shown below:

1st approximation

$$T_1 = K \log \frac{P}{S} \quad \text{.....(14)}$$

2nd approximation

$$T_2 = K^{[(\log T_1)^{\phi(T)}]} \log \frac{P}{S} \quad \text{..(23)}$$

3rd approximation

$$T_3 = K^{[(\log T_2)^{\phi(T)}]} \log \frac{P}{S} \quad \text{....(23)}$$

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

$T_3 = T$ = required thickness.

3. Sample calculations for obtaining required thickness T by

means of equation (23) for an airplane wheel loading on the basis of the following data:

Applied Load P = 100,000 lb
Subgrade Support S = 20,000 lb
Base Course Constant K = 65
$\phi(T) = 0.06$

1st approximation

$$T_1 = K \log \frac{P}{S}$$

$$T_1 = 65 \log \frac{100,000}{20,000}$$

$$T_1 = 45.43 \text{ in.}$$

2nd approximation

$$T_2 = K^{[(\log T_1)^{\phi(T)}]} \log \frac{P}{S}$$

$$T_2 = 65^{[(\log 45.43)^{0.06}]} \log\left(\frac{100,000}{20,000}\right)$$

$$T_2 = 51.7 \text{ in.}$$

3rd approximation

$$T_3 = K^{[(\log T_2)^{\phi(T)}]} \log \frac{P}{S}$$

$$T_3 = 65^{[(\log 51.7)^{0.06}]} \log\left(\frac{100,000}{20,000}\right)$$

Therefore for design $T_3 = 52.1$ in.

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

4. Sample calculation for obtaining required thickness T by means of equation (23) for a highway wheel loading on the basis of the following data,

Applied Load P = 12,000 lb
Subgrade Support S = 7,000lb
Base Course Constant K = 65
$\phi(T) = 0.06$

1st approximation

$$T_1 = K \log \frac{P}{S}$$

$$T_1 = 65 \log \left(\frac{12,000}{7,000}\right)$$

$$T_1 = 15.22 \text{ in.}$$

2nd approximation

$$T_2 = K^{[(\log T_1)^{\phi(T)}]} \log \frac{P}{S}$$

$$T_2 = 65^{[(\log 15.22)^{0.06}]} \log\left(\frac{12,000}{7,000}\right)$$

$$T_2 = 15.87 \text{ in.}$$

3rd approximation

$$T_3 = K^{[(\log T_2)^{\phi(T)}]} \log \frac{P}{S}$$

$$T_3 = 65^{[(\log 15.87)^{0.06}]} \log\left(\frac{12,000}{7,000}\right)$$

$$T_3 = 15.93 \text{ in.}$$

Therefore for design $T_3 = T = 16$ in.

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

5. The sample calculations of Items 3 and 4 above indicate that when designing for the thickness of flexible pavements for highways, or for airplane wheel loadings for which moderate thicknesses of base course are indicated, the thickness requirement will probably be given with sufficient accuracy by equation (14):

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

It would seem that only for airplane wheel loadings of about 40,000 to 50,000 lb 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 underdesign, would the use of equations (23) or (24) become necessary.

$$T = K^{[(\log T)^{\phi(T)}]} \log \frac{P}{S} \quad (23)$$

$$T = B \log \frac{P}{S} \quad \ldots\ldots\ldots\ldots (24)$$

The development of a general method for determining the required thickness of flexible pavements, presented in Item 1 above, which is

TABLE 12

INFLUENCE OF THE MAGNITUDE OF $\phi_1(T)$ ON THE VALUES OF $B = K\ (\log T)^{\phi_1(T)}$ FOR THE THICKNESSES INDICATED, WHEN THE VALUE OF K IS TAKEN AS 65 FOR THE PURPOSE OF ILLUSTRATION.

Thickness T	Values of $B = K\ (\log T)^{\phi_1(T)}$, when $\phi_1(T)$ varies from 0 to 6.0										
	$\phi_1(T)$ Values										
	0	0.2	0.4	0.6	0.8	1.0	2.0	3.0	4.0	5.0	6.0
in.											
5	65	64.5	64.1	63.6	63.2	62.7	60.5	58.4	56.3	54.3	52.4
10	65	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0
20	65	68.5	72.2	76.1	80.2	84.6	110.0	143.1	186.2	242.3	315.2
30	65	70.3	76.0	82.1	88.8	96.0	141.8	209.5	309.4	457.1	675.2
40	65	71.4	78.5	86.2	94.8	104.1	166.8	267.3	428.2	686.0	1099
50	65	72.3	80.3	89.3	99.3	110.4	187.6	318.8	541.6	920.1	1563
60	65	72.9	81.8	91.8	103.0	115.6	205.5	365.4	649.8	1155.5	2055
70	65	73.5	83.0	93.9	106.1	119.9	221.3	408.3	753.3	1390.0	2565
80	65	73.9	84.1	95.6	108.8	123.7	235.4	448.0	852.6	1622.6	3088
90	65	74.3	85.0	97.2	111.1	127.0	248.2	485.1	948.0	1852.7	3621
100	65	74.7	85.8	98.5	113.2	130.0	260.0	520.0	1040.0	2080.0	4160

summarized in equations (22), (23), and (24), might very logically have resulted in the somewhat similar series of equations outlined below:

$$T = K[f(T)^{\phi_1(T)}] \log \frac{P}{S} \quad \ldots\ldots (26)$$

or if

$$f(T) = \log T$$

then (26) becomes

$$T = K[(\log T)^{\phi_1(T)}] \log \frac{P}{S} \ldots\ldots (27)$$

or

$$T = B \log \frac{P}{S} \ldots\ldots\ldots\ldots (28)(24)$$

where

$$B = K[(\log T)^{\phi_1(T)}] \quad \ldots\ldots\ldots (29)$$

It will be observed that equations (28) and (24) are identical although derived by different methods.

In equation (26) and (27), the expression $\phi_1(T)$ will have a different set of values, other things being equal, than has the expression $\phi(T)$ of equations (22) and (23).

The influence of thickness T on the value of the expression $B = K[(\log T)^{\phi_1(T)}]$ is shown in Table 12 for a wide range of values of $\phi_1(T)$ when the value of K is taken as 65 for the purpose of illustration.

It is evident from the data of Table 12 that values of

$$B = K[(\log T)^{\phi_1(T)}]$$

of equations (26) and (27) deviate little from the value of K in equation (14) for small values of $\phi_1(T)$. For values of $\phi_1(T)$ greater than about 0.5 on the other hand, the value of $B = K[(\log T)^{\phi_1(T)}]$ may depart widely from the corresponding value of K from equation (14).

It will be observed that $\phi_1(T)$ has a constant value for each ver-

tical calumn of data in Table 12. If $\phi_1(T)$ should vary with the thickness T as its symbol permits, the influence of thickness T on the value of $\phi_1(T)$ can be inferred by reading the data of Table 12 from top to bottom in a generally diagonal direction.

It should be noted particularly, that when $\phi_1(T) = 0$, equations (26), (27), and (28) become identical with equation (14).

7. Whether general design equation (23) or equation (27) is desired for design may depend upon personal preference.

In equation (23)

$$T = K[(\log T)^{\phi(T)}] \log \frac{P}{S} \quad \ldots . \quad (23)$$

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 $K[(\log T)^{\phi(T)}]$ which depends upon the composition, moisture content, density, and thickness of the base course material

In equation (27)

$$T = K[(\log T)^{\phi_1(T)}] \log \frac{P}{S} \quad \ldots . \quad (27)$$

the right hand side provides the same information as the right hand side of equation (23), but requires four terms to do so instead of three,

(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 constant K, which depends upon the composition, moisture content, and density of the base course material

(d) The thickness factor

$$(\log T)^{\phi_1(T)}$$

which varies with the depth of base course, and therefore serves to modify the value of the base course constant K so as to make it applicable to any thickness of a given base course.

It is to be observed from Tables 11 and 12 that for a given range of values of B over the range of thickness between 10 and 100 in., the corresponding value of B for a thickness of 5 in. is much less in Table 11 (equation 23) than in Table 12 (equation 27).

8. It was pointed out in Item 5 above that for highway wheel loadings, and for moderate thickness requirements for airplane wheel loadings, the required thickness seems to be given with sufficient accuracy by equation (14). Only for the heavier airplane wheel loads, which must be carried on runways with low subgrade support, does the use of equations (23) or (27) appear to be indicated since for these cases equation (14) might give thicknesses that are too small and therefore lead to underdesign.

On the other hand, as a result of their investigations, the U.S. Corps of Engineers has 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 airplane wheel loadings than for smaller wheel loadings(20).

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. 104 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 in-

creased (Fig. 25). If the subgrade support S is increased for any given applied load P, the value of the expression $\log \frac{P}{S}$ becomes smaller. It is generally true that the larger values of $\log \frac{P}{S}$ apply to the wheel loadings of heavier airplanes. 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. 104 would tend to approach the straight line 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, equation (14) (based upon a constant deflection throughout) may have a wider range of application than the previous development in this section would suggest.

9. For graphs of equations (14), (23), (27), etc. which have been shown in a number of the diagrams for this paper, a constant deflection for both P and S in the expression $\log \frac{P}{S}$ has to be assumed over the whole range employed in each case. It is to be emphasized however, that these equations are equally applicable if a varieble 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 different shape. Consequently, equations (14), (23), (27), 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 (tire imprint area), as suggested by the investigation of the USED.

For the sake of clarity however, it might be preferable to consider that when P and S of the expression $\log \frac{P}{S}$ appear without subscripts, as in the present notation for equations (14), (23), (27), etc. a constant critical deflection, e.g. 0.5 in., 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

$$T = K \log \left(\frac{P_d}{S_d}\right) \quad \ldots\ldots\ldots \quad (30)$$

equation (23) would be written as

$$T = K\left[(\log T)^{\phi(T)}\right] \log \left(\frac{P_d}{S_d}\right) \ldots \quad (31)$$

equation (27) would be written as

$$T = K\left[(\log T)^{\phi_1(T)}\right] \log \left(\frac{P_d}{S_d}\right) \ldots \quad (32)$$

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.

CONCLUSION

In probably every investigation of this character, the need is felt for further data to check the trends that seem to have been developed, or to explore toward the wider horizons which have been revealed. The accuracy of the various figures, and of the findings tentative or otherwise, that have been presented in the foregoing pages, has been

limited by the data obtained during our investigation, supplemented by pertinent information published by other investigators. It is to be expected therefore, that further investigation will reveal the necessity for some modification of a number of the diagrams, and for some revision of the conclusions which have been pointed out, since this is a normal occurrence.

SUMMARY

1. This paper outlines the results of an investigation which has been conducted by the Department of Transport during 1945 and 1946 at a number of Canadian airports.

2. Traffic experience at several of Canada's busier airports indicated that the current flexible pavement thickness requirements of several principal organizations in the U.S.A. for runways, are ultra conservative.

3. A pedological soil survey of an airport site provides valuable information by indicating the areas of subgrade which have different engineering properties.

4. Field moisture and density data demonstrate that at only a small number 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. Traffic information versus load test data at Canadian airports indicates that safe runway design can be based upon a deflection of 0.5 in. after 10 repetitions of load.

7. For any given deflection for plate bearing tests on cohesive soils, a straight line relationship seems to hold for unit load support versus the $\frac{P}{A}$ ratio of a series of steel bearing plates over the range of bearing plate diameters between 12 and 42 in., and probably beyond.

8. Ratios have been developed between the magnitudes of load supported at 1, 10, and 100 repetitions for any given deflection between 0.2 and 0.5 in., all other factors being equal.

9. A study of the load versus deflection curves has indicated that if the load supported at 0.2-in. deflection on a 30-in. 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 and 0.7 in. for bearing plates between 12 and 42 in. in diameter, and probably beyond, may be calculated.

10. The average yield point deflection for subgrades of the airports included in the investigation seems to occur at 0.26 in., and the average yield point deflection for the bituminous surfaces appears to be 0.225 in.

11. Base course support per unit of thickness may be generally independent of the composition of granular base course materials, but seems to be influenced by base course density. Further study may indicate that composition, moisture content, density, and thickness may all have to be considered when determining base course support per unit thickness.

12. Bituminous surfaces appear 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.

13. Relationships have been

established between plate bearing test results versus cone bearing, Housel penetrometer, field CBR, and triaxial compression test data.

14. A method for designing bituminous paving mixtures and selecting base course materials by means of the triaxial compression test is outlined.

15. The load test data suggest that the subgrade modulus k for rigid pavement design should be determined under repetitional rather than static loading, and with bearing plates considerably larger than that of 30 in. in diameter in common use at the present time.

16. 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. The supporting value of the subgrade must be determined for the same diameter of bearing plate, for the same number of repetitions of load, and for the same deflection as the applied surface load.

17. A method of design giving the required thickness of granular base for supporting wheel loads of any magnitude has been developed from the load test data on surface and subgrade.

18. 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 airplane wheel loadings. One set of curves is based upon plate bearing tests, and another set on cone bearing, Housel penetrometer, field CBR, and triaxial compression tests.

19. Load test data were obtained to demonstrate the influence of dual versus single tires on runway design.

20. Charts of thickness design curves for flexible pavements for highway wheel loadings have also been prepared, based upon plate bearing tests, and upon cone bearing, Housel penetrometer, field CBR, and triaxial compression tests.

21. General equations of design for required thickness of flexible pavements have been developed, based upon applied load, subgrade support, and base course support per unit of thickness.

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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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APPENDIX

SAMPLE CALCULATIONS

A number of sample calculations are worked out below for the evaluation and design of flexible pavements for airport runways and taxiways, etc., and for highways. These illustrations are based upon the data assembled in the previous pages which were derived from the Department of Transport's investigation, and in particular upon equation (15),

$$T = 65 \log \frac{P}{S} \quad \text{.....} \quad (15)$$

It will be recalled that K in equation (14) was found to have the average value of 65 for the base course data obtained so far. With the advent of much heavier compaction equipment, higher densities may be obtained for base courses than has been the case in the past, and this in turn may justify a lower value than 65 for K. At the present time however, a value of $K = 65$ as indicated for equation (15) is recommended as being most representative, and is employed for the sample calculations which follow.

The critical deflection for both runway and highway design employed in these illustrative examples is 0.5 in. after 10 repetitions of load. This provides thicknesses of granular base course which experience indicates to be reasonable. Greater or lesser thicknesses of granular base are obtained if the critical deflection is assumed to be smaller or larger respectively than 0.5 in.

For taxiway, apron, and turnaround design, the critical deflection employed in the following sample calculations is 0.225 in. after 10 repetitions of load. If indicated to be necessary after greater experience, the thickness of granular base required for taxiways, aprons, or turnarounds, can be increased or decreased by employing a smaller or larger critical deflection, respectively, than 0.225 in.

Capacity operations for runway and taxiway design are based upon 10 repetitions of load for deflections of 0.5 and 0.225 in., respectively, for these sample calculations. For either limited operations or intermediate landing fields, runway and taxiway designs are based upon one application of load for these deflections. Table 5 indicates that for the same contact area, the load carried for one application is 15 percent greater than that supported at 10 repetitions over the deflection range from 0.2 to 0.5 in. This ratio has been found to apply to both cohesive subgrades and bituminous surfaces.

AIRPORT RUNWAYS, TAXIWAYS, APRONS, ETC.

EVALUATING THE LOAD SUPPORTING CAPACITY OF A FLEXIBLE PAVEMENT

Sample Calculation No. 1

(a) The following load test data have been obtained at eight locations on the surface of a paved runway at the most critical time of year, for a 30-in. plate, at 0.5-in. deflection, after 10 repetitions of load. What is the maximum safe wheel loading for this runway for,

(1) Capacity operations

(2) Emergency landings or limited operations?

If the load test data apply to a taxiway, apron, or turnaround, what is the maximum safe wheel load for,

(1) Capacity operations

(2) Emergency landings or limited operations?

Total Load in Pounds
38200
37600
35400
33800
33100

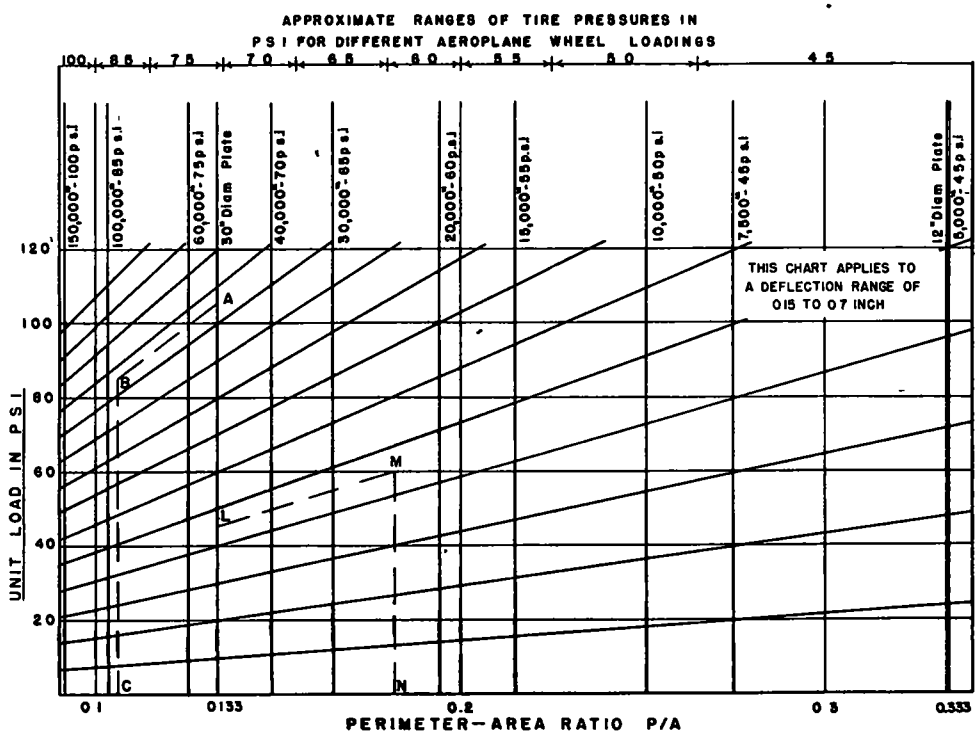

Fig. A Diagram for Evaluating Load Carrying Capacity of Runways with Flexible Pavements (Wheel Load on Single Tire)

Total Load in Pounds

32200
Lower quartile point --------- 31800
31400
30900

Step No. 1-The lower quartile point is higher than 25 percent of the data, but less than 75 percent, and is taken as the representative load test value. The lower quartile point for the load test data for this runway is 31,800 lb, or a unit load of 45 psi, since the area of a 30-in. plate is 707 sq in.

Step No. 2-The $\frac{P}{A}$ ratio for a 30-in. bearing plate is 0.133.

In general, as airplane wheel loads increase, tire pressures and contact areas increase, and the $\frac{P}{A}$ ratios of the contact areas therefore decrease. Figure A is a graph of unit load in psi versus $\frac{P}{A}$ ratio. Across the top of the graph, the airplane tire pressures which are normal for each range of $\frac{P}{A}$ ratios are given.

It will be observed in connection with Fig. 39, that for surface load tests the ratio of the unit load on a 12-in. plate to that supported by a 30-in. plate over the range of deflection from 0.2 to 0.5 in., is about 2.45. The oblique lines on Fig. A are drawn on the basis that the ratio of the unit load on a 12-in. plate versus that on a 30-in. plate is 2.4.

The data across the top of Fig. A demonstrate that a unit pressure of 45 psi and a $\frac{P}{A}$ ratio of 0.133 (30-in. plate) do not correspond. The problem therefore is to find the smaller contact area (larger $\frac{P}{A}$ ratio) and the correspondingly higher unit pressure, which are usually associated with each other.

Point *L* in Fig. A represents a unit load of 45 psi for a 30-in. plate ($\frac{P}{A} = 0.133$). The broken line *LM* is drawn to the right approximately parallel to the two nearest oblique lines. Its actual slope is such that the ratio of the unit load on a 12-in. plate to that on a 30-in plate is 2.4. Line *LM* is projected to *M*, the coordinates of

which indicate that the unit pressure on the left hand side of the graph, 60 psi, is equal to the tire pressure within one of the ranges shown across the top of the chart, 60 psi.

The vertical broken line *MN* through *M* cuts the abscissa at the required $\frac{P}{A}$ ratio, 0.181.

Step No. 3-The total load corresponding to a tire pressure of 60 psi, and a $\frac{P}{A}$ ratio of 0.181, can be calculated, but may be more readily obtained from Fig B

the same contact area, the load carried at 0.5-in. deflection for one loading is 15 percent greater than the load supported at 0.5 in. deflection for 10 repetitions.

The lower quartile point for the listed data at one loading is therefore (1.15)(31,800) = 36,600 lb, or a unit load of 51.8 psi.

Step No. 5-By applying the procedure of Steps No. 2 and 3, Fig. A indicates that a unit load of 51.8 psi on a 30-in. plate corresponds to a tire pressure of 65 psi at a $\frac{P}{A}$ ratio of 0.1694. From

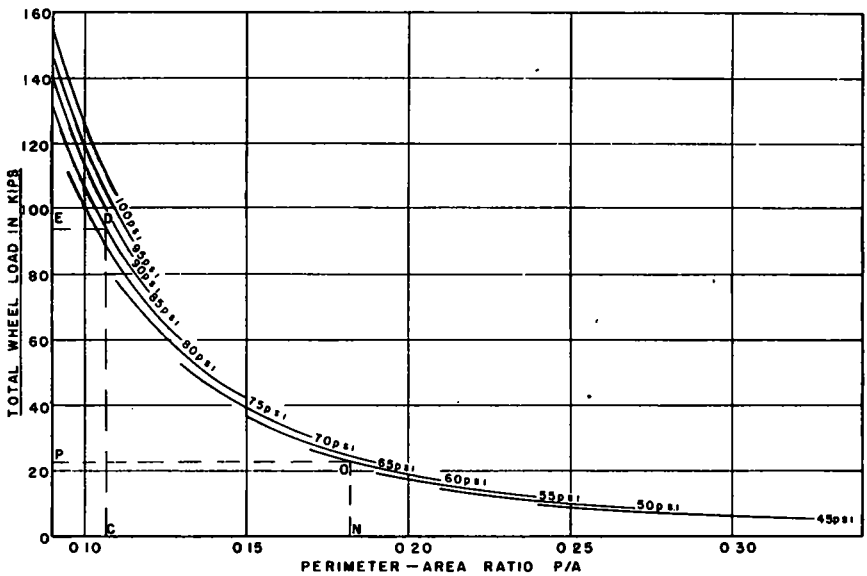

Fig. B Diagram for Evaluating Load Carrying Capacity of Runways with Flexible Pavements (Wheel Load on Single Tire)

From Point *N* at the $\frac{P}{A}$ ratio of 0.181, in Fig. B, the dashed vertical line *NO* is drawn to intersect the tire pressure curve labelled 60 psi at *O*. From *O* the horizontal broken line *OP* is drawn to cut the ordinate axis at *P*, representing the required total load, 23,000 lb.

Therefore, the maximum wheel load which this runway can carry for capacity operations is 23,000 lb.

Step No. 4-Runway design for an emergency or intermediate landing field, or for limited operations, can be based upon a deflection of 0.5 in. for one loading Table 5 indicated that for

Fig. B, the total load corresponding to a $\frac{P}{A}$ ratio of 0.1694 and a tire pressure of 65 psi is seen to be 28,400 lb.

Therefore, the maximum safe wheel load for this runway if used for emergency landings or limited operations is 28,400 lb.

Step No. 6-Taxiway, apron, and turnaround design for capacity operations is based upon a deflection of 0.225 in. after 10 repetitions of load.

The ratios of Fig 33 for a 30-in. plate indicate that a load of 31,800 lb at 0.5-in. deflection corresponds to a

$$\text{load of } \frac{(31{,}800)(1.075)}{(1.633)} = 20{,}950 \text{ lb, or}$$

29 6 psi at 0.225-in deflection.

Step No 7—According to the procedure of Steps No 2 and 3, Fig. A demonstrates that a unit load of 29.6 psi on a 30-in. plate corresponds to a tire pressure of 50 psi at a $\frac{P}{A}$ ratio of 0.2365. From Fig B, the total load corresponding to a $\frac{P}{A}$ ratio of 0.2365 and a tire pressure of 50 psi is seen to be 11,000 lb

Consequently, the maximum safe wheel load for this taxiway, apron, or turnaround for capacity operations is 11,000 lb

Step No. 8—Taxiway, apron, and turnaround design for emergency landing fields or limited operations can be based upon load supported at 0.225-in. deflection after one application of load.

The lower quartile point for the above data for a 30-in. plate at one application of load is (1.15)(31,600) equals 36,600 lb (Table 5).

The ratios of Fig. 33 for a 30-in. plate indicate that a load of 36,600 lb at 0.5-in. deflection corresponds to a

$$\text{load of } \frac{(36{,}600)(1.075)}{(1\ 633)} = 24{,}100 \text{ lb or}$$

34.1 psi at 0.225-in deflection.

Step No 9—According to the procedure of Steps No. 2 and 3, Fig. A indicates that a unit load of 34.1 psi on a 30-in. plate corresponds to a tire pressure of 55 psi, at a $\frac{P}{A}$ ratio of 0.221. From Fig. B, the total load corresponding to this $\frac{P}{A}$ ratio and a tire pressure of 55 psi is 14,200 lb. Therefore, the maximum safe wheel loading for this taxiway, apron, or turnaround for emergency or limited operations is 14,200 lb.

(b) If the load test data for a runway were as follows, with other conditions identical with those for (a) above, what is the maximum safe wheel load for this runway for,

(1) Capacity operations

(2) Emergency landings or limited operations?

If the load test data apply to a taxiway, apron, or turnaround, what is the maximum safe wheel load for,

(1) Capacity operations

(2) Emergency landings or limited operations?

Total Load in Pounds

87600
81900
78800
78300
77200
75500
Lower quartile point ------- 74300
73200
68400

Step No. 1—The lower quartile point occurs at a total load of 74,300 lb, or a unit load of 105 psi.

Step No. 2—For reasons similar to those outlined in (a), above, and from dashed lines *AB* and *BC* of Fig. A, and *CD* and *DE* of Fig. B, the maximum safe wheel load on a single tire for capacity operations for this runway is 93,000 lb.

Step No. 3—The lower quartile point for the listed data for one application of load is (1.15)(74,300) = 85,450 lb, or a unit load of 121 psi.

Step No 4—By applying the procedure of Step No 2, Fig. A indicates that a unit load of 121 psi on a 30-in. plate corresponds to a unit pressure of 93 psi at a $\frac{P}{A}$ ratio of 0.1.

It is to be noted that the ranges of tire pressures across the top of the graph for the corresponding $\frac{P}{A}$ ratios shown in Fig. A are approximate only. Considerable overlapping of these ranges may occur for the wheels of different aircraft. In Fig. A, the range of tire pressures indicated increases abruptly from 85 to 100 psi at a $\frac{P}{A}$ ratio of 0.1. Consequently, for the range of unit loads between 85 and 100 psi along the left hand side, there is no corresponding range of tire pressures across the top of the graph. It is believed that in this case sufficient accuracy will result if the required total load is calculated on the basis of the unit pressure indicated on the left hand side of the graph, at

the intersection of the oblique line in question (approximately parallel to *AB*) with the 0.1 value for the $\frac{P}{\lambda}$ ratio.

From Fig. B, the total load corresponding to a $\frac{P}{\lambda}$ ratio of 0.1 and a tire pressure of 93 psi is 118,000 lb (single tire).

Therefore, the maximum safe wheel load for this runway, if used for emergency landings or limited operations is 118,000 pounds (single tire).

Step No. 5-The ratios of Fig. 33 for a 30-in. plate indicate that a load of 74,300 lb at 0.5-in. deflection corresponds to a load of $\frac{(74,300)(1.075)}{(1.633)}$ equals 48,900 lb, or 69.1 psi at 0.225-in. deflection.

Step No. 6-Following the procedure of Step No. 2, Fig. A indicates that a unit load of 69.1 psi on a 30-in. plate corresponds to a tire pressure of 70 psi at a $\frac{P}{\lambda}$ ratio of 0.1352. From Fig. B, the total load corresponding to a $\frac{P}{\lambda}$ ratio of 0.1352 and a tire pressure of 70 psi is 48,000 lb (single tire).

Consequently, the maximum safe wheel load for this taxiway, apron, or turnaround, for capacity operations is 48,000 lb (single tire).

Step No. 7-The lower quartile point for the load test data for a 30-in. plate for one load is (1.15)(74,300) equals 85,450 lb.

The ratios of Fig. 33 for a 30-in. plate indicate that a load of 85,450 lb at 0.5-in. deflection corresponds to a load of $\frac{(85,450)(1.075)}{(1.633)} = 56,200$ lb, or 79.5 psi at 0.225-in. deflection.

Step No. 8-Following the procedure of Step No. 2, Fig. A indicates that a unit load of 79.5 psi on a 30-in. plate corresponds to a tire pressure of 75 psi at a $\frac{P}{\lambda}$ ratio of 0.1252. From Fig. B the total load corresponding to a $\frac{P}{\lambda}$ ratio of 0.1252 and a tire pressure of 75 psi is 60,000 lb (single tire). Therefore, the maximum safe wheel load for this taxiway, apron, or turnaround, for emergency of limited operations is 60,000 lb.

NOTE: It will be noted that the wheel loads determined in Steps No. 2, 4, 6, and 8 above, are for a single tire Wheel loads of the magnitude indicated by each of these steps would ordinarily be carried on dual tires. With dual tires, a wheel load up to 35 percent greater than that shown in each case, depending upon the thickness of base and surface, could be safely supported.

Sample Calculation No. 2

The runway pavement consists of 4 in. of asphaltic concrete, and 20 in. of granular base course. No load test equipment was available, but the following cone bearing and Housel penetrometer data were obtained at eight locations for the top two feet of the cohesive soil subgrade under the pavement at the most critical time of the year. The pavement is seven years old, and equilibrium conditions of moisture have had ample time to develop in the subgrade. What is the maximum safe wheel load for this runway for,

(1) Capacity operations

(2) An emergency or intermediate field, or for limited operations?

If the test data apply to a taxiway, apron, or turnaround, what is the maximum safe wheel load for,

(1) Capacity operations

(2) An emergency or intermediate field, or for limited operations?

Cone Bearing Data psi	*Housel Penetrometer Data - Number of Blows for a penetration of 6 in.*
560	38
510	33
485	32
460	30
410	28
400	27
Lower quartile point ------ 390	---------- 26
380	25
365	22

Step No. 1-The lower quartile values are 390 psi for the cone bearing data, and 26 blows for the Housel pentro-

meter tests

Step No. 2-If base and surface are considered equivalent in load supporting value to 24 in. of granular base, Figs 95 and 97 indicate that for a cone bearing value of 390 psi, the load carrying capacity of the runway is 90,000 lb. They also show that for a Housel penetrometer value of 26 blows, the load carrying capacity of the runway is 110,000 lb.

It has been shown (text) that the subgrade bearing capacity obtained by averaging the ratings provided by the cone bearing and Housel penetrometer tests, gave a result within 10 percent of that obtained by load testing. On this basis, the maximum safe wheel load for capacity operations on this runway is 100,000 lb (single tire).

Step No. 3-For a wheel load of 100,000 lb, and a tire pressure of 85 psi, the contact area is 1176 sq in., and the $\frac{P}{A}$ ratio of the equivalent circular contact area is 0.1034.

The wheel load carried on the same contact area at one application of load is (1.15)(100,000) = 115,000 lb, or a unit load of $\frac{115,000}{1176}$ = 97.8 psi.

Step No. 4-A tire pressure of 97.8 psi is greater than that associated with a $\frac{P}{A}$ ratio of 0.1034 in Fig A.

Following the procedure of Steps No. 2 and 3 of *Sample Calculation No. 1* (a) and of Step No. 4 of *Sample Calculation No. 1* (b), Fig. A indicates that a unit load of 97.8 psi on a contact area of 1176 sq in. ($\frac{P}{A}$ = 0.1034) corresponds to a unit pressure of 95.5 at a $\frac{P}{A}$ ratio of 0.1.

From Fig. B, the total load corresponding to a $\frac{P}{A}$ ratio of 0.1 and a tire pressure of 95.5 psi is 120,000 lb (single tire).

Therefore, the maximum safe wheel load for this runway, if used for emergency landing or limited operations is 120,000 lb (single tire).

Step No. 5-If the thickness of base and surface is equivalent to 24 in. of granular base, Fig. 97 demonstrates that for a cone bearing value of 390 psi the load carrying capacity of a taxiway, apron, or turnaround is 42,000 lb, and for a Housel penetrometer rating of 26 blows is 51,000 lb. Averaging these two values gives a load carrying capacity of 46,500 lb.

Consequently, the maximum safe wheel load for this taxiway, apron, or turnaround for capacity operations is 46,500 lb (single tire)

Step No. 6-Figure A indicates a tire pressure of 70 psi for a wheel load of 46,500 lb. The corresponding contact area is 665 sq in., and the $\frac{P}{A}$ ratio is 0.137.

The wheel load carried on the same contact area at one application of load is (1.15)(46,500) = 53,480 lb, or a unit load of $\frac{53,480}{665}$ = 80.5 psi.

Step No. 7-A tire pressure of 80.5 psi is greater than that associated with a $\frac{P}{A}$ ratio of 0.137 in Fig. A.

Following the procedure of Step No. 4 immediately above, Fig. A indicates that a unit load of 80.5 psi on a contact area of 664 sq in. ($\frac{P}{A}$ = 0.137) corresponds to a unit pressure of 75 psi at a $\frac{P}{A}$ ratio of 0. 128.

From Fig B, the total load corresponding to a $\frac{P}{A}$ ratio of 0.128 and a tire pressure of 75 psi is 58,000 lb

Therefore the maximum safe wheel load for this taxiway, apron, or turnaround for emergency landings or limited operations is 58,000 lb (single tire).

NOTE: Wheel loads of the magnitude indicated by Steps No. 2, 4, 5, and 7, would ordinarily be carried on dual tires With dual tires, a wheel load up to 35 percent greater than that shown in each case, depending upon the thickness of base and surface, could be safely supported.

It was found during the investigation that 1 in. of properly designed and constructed asphaltic concrete containing asphaltic cement harder than 100 penetration, has the load supporting value of 2.5 in. of granular base. If this is not utilized as a safety factor, the equivalent thickness of granular base course in *Sample Calculation No. 2* becomes 30 in., made up as follows,

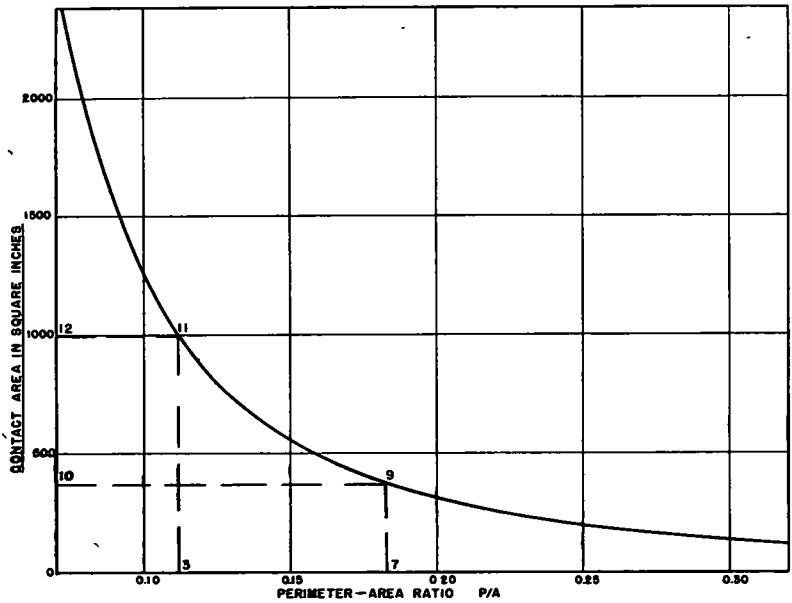

Fig. C Contact Area Versus Perimeter-Area Ratio for Airplane Wheel Loadings

Thickness of base course	20 in.
Equivalent thickness of 4 in. of asphaltic concrete	10 in.
Equivalent total thickness of granular base	30 in.

Unless the additional strength of the asphaltic concrete is utilized as a safety factor, therefore, Steps No. 1 to 7 could be worked out on the basis of 30 in. of granular base course rather than 24 in.

For runways surfaced with bituminous pavements containing liquid asphalt or soft asphalt cement binders, the investigation indicated that 1 in. of bituminous surface has the load supporting capacity of 1.5 in. of granular base.

It should be added by way of a precautionary comment, however, that there is some evidence that a new bituminous pavement does not have any greater supporting value per unit of thickness than granular base course material. The greater supporting capacity of the bituminous surface does not seem to develop until after it has been exposed to traffic for a time.

In tropical and semi-tropical climates, because of higher average pavement temperatures, the supporting value of a bituminous surface per unit of thickness should not be taken to be any greater than that of granular base course material, unless this has been proven by plate bearing tests on actual projects in the field.

FLEXIBLE PAVEMENT DESIGN FOR AIRPORT RUNWAYS AND TAXIWAYS, ETC.

Sample Calculation No. 3

(a) When plate bearing tests (30 in. in diameter, 0.5-in. deflection, 10 repetitions of load) on a cohesive subgrade in its most critical condition give the following data for eight representative locations, what thickness of granular base is required to carry an airplane wheel load of 22,500 lb at a tire pressure of 60 psi on a runway for,

(1) Capacity operations

(2) Emergency landings or limited operations?

With all other conditions as outlined above, what thickness of granular base is required for a taxiway, apron or turnaround for,

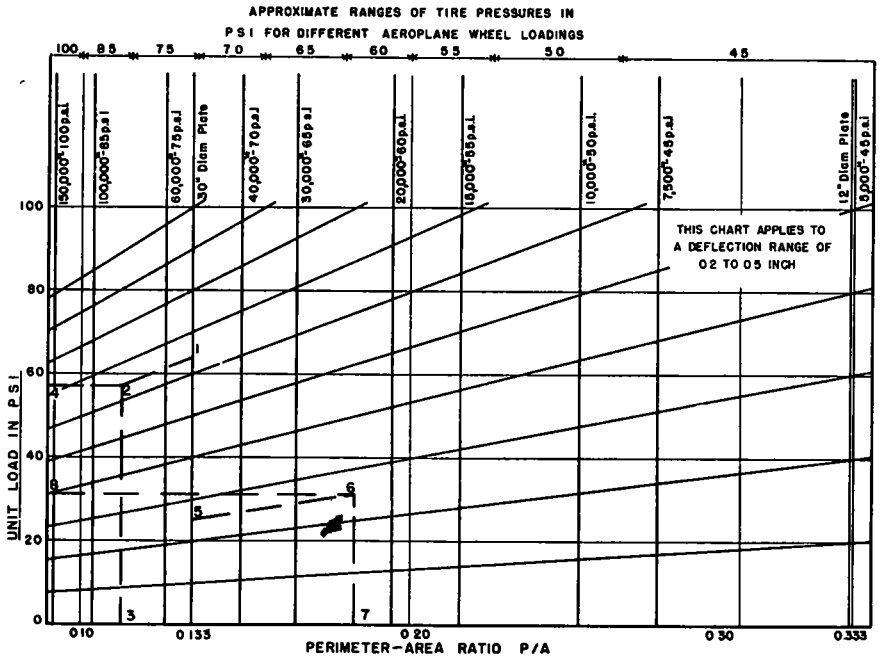

Fig. D Diagram for Evaluating Load Carrying Capacity of Subgrades for Airport Runways (Wheel Load on Single Tire - Flexible Pavements)

(1) Capacity operations

(2) Emergency landings or limited operations?

Total Load in Pounds	
21200	
19500	
19000	
18700	
18200	
18000	
Lower quartile point ----------	17700
17400	
16900	

Step No. 1-The lower quartile point occurs at a value of 17,700 lb. For a 30-in. diameter plate this corresponds to a unit pressure of 25 psi.

Step No. 2-For a wheel load of 22,500 lb and a tire pressure of 60 psi the contact area is 375 sq in. From Fig. C, the $\frac{P}{A}$ ratio corresponding to this contact area is 0.183.

Step No. 3-It is required to determine the unit load for a contact area of 375 sq in., ($\frac{P}{A}$ ratio = 0.133). Figure D is employed for this purpose.

For cohesive subgrades, Fig. 37 demonstrates that the unit load supported on a 12-in. plate is nearly 2.1 times that on a 30-in. plate over a deflection range of 0.2 to 0.5 in. The oblique lines in Fig. D are drawn on the basis that the unit subgrade support on a 12-in. plate is twice that on a 30-in. plate for the same range of deflection. In Fig. D, Point 5 represents a subgrade support of 25 psi on a 30-in. plate at 0.5-in deflection. The vertical dashed line *7,6* is drawn through the $\frac{P}{A}$ value of 0.183. Broken line *5,6* is drawn to the right from Point 5 approximately parallel to the nearest oblique lines. The actual slope of line *5,6* is such that the unit load on a 12-in. plate is twice that for a 30-in. plate. From Point 6, at the intersection of dashed lines *5,6* and *7,6*, the horizontal broken line *6,8* is drawn, which cuts the ordinate axis at a unit pressure of 31.25 psi. That is,

a unit pressure of 31.25 psi for a $\frac{P}{S}$ ratio of 0 183 (contact area of 375 sq in) is the required value of subgrade support corresponding to 25 psi on a 30-in. plate. Therefore, the load supporting value of the subgrade for a contact area of 375 sq in. (375)(31.25) = 11,720 lb.

Step No. 4-To carry a wheel load of 22,500 lb on a runway for which the subgrade support at 0.5-in. deflection is 11,720 lb, a base course thickness of 18 in is indicated by interpolation in Fig. 96.

To avoid the possible inaccuracies of interpolation, or as an additional check, the base course thickness requirement may also be obtained directly by substitution in equation (15):

$$T = 65 \log \frac{P}{S}$$

$$= 65 \log \left(\frac{22500}{11720}\right)$$

$$= 18.4 \text{ in.}$$

Therefore, a runway constructed on this subgrade will require 18 in. of granular base or its equivalent, for capacity operations by aircraft with wheel loads of 22.500 lb.

Step No. 5-The lower quartile point for the listed data for one application of load is (1.15)(17,700) = 20,350 lb or 28.8 psi

Step No. 6-Following the procedure of Steps No 2, 3, and 4, Fig. D indicates that a unit pressure of 36 psi is the required value of subgrade support corresponding to 28.8 psi on a 30-in. plate. Therefore, the load supporting value of the subgrade at 0.5-in. deflection for one application of load on a contact area of 375 sq in. equals (375)(36) = 13,500 lb

Step No. 7-To carry a wheel load of 22,500 lb on a runway for which the subgrade support at 0.5-in. deflection is 13,500 lb, Fig. 96 indicates that 14 in. of granular base are required.

The thickness given by equation (15) is

$$T = 65 \log \left(\frac{22,500}{13,500}\right)$$

$$= 14.4 \text{ in.}$$

Therefore, a runway constructed on this subgrade will require 14 in. of granular base course or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 22,500 lb.

Step No 8 - When the subgrade support is 11,720 lb, the base course thickness requirement for a taxiway, apron, or turnaround, to carry a wheel load of 22,500 lb, is indicated by interpolation in Fig. 96 to be 29 in.

Since Fig. 27 demonstrates that for a bearing plate of any given size, there is a definite ratio between the load carried at 0.5-in. to that supported at 0.225-in. deflection, the required thickness of granular base for taxiways, aprons, and turnarounds can be determined by means of equation (15). If the subgrade support is 11,720 lb at 0.5-in. deflection, Fig. 27 indicates it will be $\frac{(1.075)(11,720)}{(1.585)} = 7,950$ lb, at a deflection of 0.225 in. Therefore the thickness of granular base required for a taxiway, apron, or turnaround is given by

$$T = 65 \log \left(\frac{22500}{7950}\right)$$

$$= 29.4 \text{ in.}$$

Consequently, a taxiway, apron, or turnaround on this subgrade will require 29 in. of granular base, or its equivalent, for capacity operations by aircraft with wheel loads of 22,500 lb.

Step No. 9-If the subgrade support for the required contact area is 11,720 lb at 0.5-in. deflection for 10 repetitions, it is (1.15)(11720) equals 13,480 lb for one loading.

Step No. 10-When the subgrade support is 13,480 lb, the base course thickness requirement for a taxiway, apron, or turnaround, to carry a wheel load of 22,500 lb is indicated by interpolation in Fig. 96 to be 25 in.

The required thickness may also be derived in another manner:

From Step No. 8, the subgrade support for the required contact area is 7,950 lb at 0.225-in deflection for 10 repetitions. Therefore, for one loading at 0.225-in. deflection the subgrade support will be (1.15)(7950) = 9,140 lb.

The thickness of granular base required to carry a wheel load of 22,500 lb when the subgrade support is 9,140 lb, is given by,

$$T = 65 \log \left(\frac{22500}{9140}\right)$$

$$= 25.4 \text{ in.}$$

Therefore, a taxiway, apron, or turnaround constructed on this subgrade will require 25 in. of granular base, or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 22,500 lb.

(b) When plate bearing tests (30-in. diameter, 0.5-in. deflection, 10 repetitions of load) on a cohesive subgrade in its most critical condition give the following data for eight representative locations, what thickness of granular base is required to carry an airplane wheel load of 85,000 lb at a tire pressure of 85 psi on a runway for,

(1) Capacity operations

(2) Emergency landings or limited operations?

With all other conditions as outlined above, what thickness of granular base is required for a taxiway, apron, or turnaround for,

(1) Capacity operations

(2) Emergency landings or limited operations?

Total Load in Pounds

54200
51300
48700
47300
47200
46300
Lower quartile point -------- 45200
44100
43600

Step No. 1-Subgrade support at the lower quartile point is 45,200 lb, or 64 psi.

Step No. 2-For a wheel load of 85,000 lb, and a tire pressure of 85 psi the contact area is 1000 sq in. From Fig C, the $\frac{P}{A}$ ratio corresponding to this contact area is 0.112.

Step No. 3-Point 1 in Fig. D represents a subgrade support of 64 psi on a 30-in. plate at 0.5-in. deflection. The dashed vertical line 3,2 is drawn at a $\frac{P}{A}$ ratio of 0.112. When broken lines 1,2 and 2,4 are drawn as shown, Point 4 on the ordinate axis gives the unit subgrade support, 57.3 psi, for a $\frac{P}{A}$ ratio of 0.112 (contact area 1000 sq in.). The subgrade support for a contact area of 1000 sq in. therefore is (57.3)(1000) = 57,300 lb.

Step No. 4-By interpolation, Fig. 96 indicates that for a subgrade support of 57,300 lb, a wheel load of 85,000 lb on a runway requires a granular base course 11 in. thick.

The thickness requirement for a runway as given directly by equation (15) is,

$$T = 65 \log \left(\frac{85,000}{57,300}\right)$$

$$= 11.1$$

Therefore, a runway constructed on this subgrade will require 11 in. of granular base or its equivalent, for capacity operations by aircraft with wheel loads of 85,000 lb.

Step No. 5-The lower quartile point for the listed data at one application of load is (1.15)(45,200) = 51,980 lb, or 73.5 psi

Step No. 6-Following the procedure of Steps No. 2, 3, and 4, immediately above, Fig. D indicates that a unit pressure of 65.8 psi for a $\frac{P}{A}$ ratio of 0.112 is the required value of subgrade support corresponding to 73.5 psi on a 30-in. plate. Therefore, the load supporting value of the subgrade at 0.5-in. deflection for one loading for a contact area of 1000 sq in. is (65.8)(1000) = 65,800 lb.

Step No. 7-To carry a wheel load of 85,000 lb on a runway for which the subgrade support at 0.5-in. deflection is 65,800 lb, Fig. 96 indicates that 7 in. of granular base are required.

The required thickness may also be obtained directly from equation (15)

$$T = 65 \log \left(\frac{85,000}{65,800}\right)$$

$$= 7.2 \text{ in.}$$

Therefore a runway constructed on this subgrade will require 7 in. of granular base or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 85,000 lb.

Step No. 8-When the subgrade support is 57,300 lb, the base course thickness requirement for taxiways, aprons, and turnarounds, provided by interpolation in Fig 96, is 23 in.

If the subgrade support is 57,300 lb at a deflection of 0.5 in., Fig. 27 indicates that it will be $\left(\frac{1.080}{1\ 650}\right)(57,300)$ equals 37,500 lb at a deflection of 0.225 in The required thickness of granular base for a taxiway, apron, or turnaround is given by equation (15)

$$T = 65 \log \left(\frac{85,000}{37,500}\right)$$

$$= 23 \text{ in.}$$

Consequently, a taxiway, apron, or turnaround on this subgrade will require 23 in. of granular base course or its equivalent, for capacity operations by aircraft with wheel loads of 85,000 lb.

Step No. 9-If the subgrade support for the required contact area is 57,300 lb at 0.5-in. deflection for 10 repetitions, it is (1.15)(57,300) = 65,800 lb for one loading.

Step No. 10-When the subgrade support is 65,800 lb, the base course thickness requirement for a taxiway, apron, or turnaround, to carry a wheel load of 85,000 lb, is indicated by interpolation in Fig. 96 to be 18 in.

The required thickness may be derived in another manner.

From Step No. 8, the subgrade support for the required contact area is 37,500 lb at 0.225-in. deflection for 10 repetitions. Therefore, for one loading at 0.225-in. deflection, the subgrade support will be (1.15)(37,500) equals 43,150 lb.

The thickness of granular base required to carry a wheel load of 85,000 lb when the subgrade support is 43,150 lb is given by equation (15)

$$T = 65 \log \left(\frac{85,000}{43,150}\right)$$

$$= 18.4 \text{ in.}$$

Therefore, a taxiway, apron, or turnaround constructed on this subgrade will require 18 in. of granular base, or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 85,000 lb.

NOTE: Notes appearing at the end of *Sample Calculation No. 2* are equally applicable to *Sample Calculation No. 3.*

Sample Calculation No. 4

If the following cone bearing and Housel penetrometer values have been obtained at eight representative locations for the top 24 in of a cohesive subgrade in its most critical condition, under a pavement, what thickness of granular base is required to carry an airplane wheel load of 60,000 lb on a runway for,

(1) Capacity operations

(2) Emergency landings or limited operations?

With all other conditions as outlined above, what thickness of granular base is required for a taxiway, apron, or turnaround for,

(1) Capacity operations

(2) Emergency landings or limited operations?

Cone bearing data psi	*Housel penetrometer data Number of blows for a penetration of 6 in.*
420	26
390	24
335	21
310	18
280	17
275	16
Lower quartile point ----- 260	-------- 15
245	14
235	12

Step No. 1-The lower quartile value for the cone bearing data is 260 psi, and for the Housel penetrometer tests is 15 blows.

Step No. 2-An airplane wheel load of 60,000 lb would ordinarily be carried on dual tires. This is about 120 percent (in this case) of the load carried on a single tire. Runway design in this case should therefore be based upon a wheel load of 50,000 lb supported on a single tire.

Step No. 3-For a subgrade cone bearing value of 260 psi and a wheel load of 50,000 lb, Fig. 97 indicates that 23 in. of granular base are required for runways. For a Housel penetrometer value of 15 blows, 25 in. of granular base are indicated. Averaging these two requirements gives a thickness of 24 in. of granular base, or its equivalent, for capacity operations by aircraft with wheel loads of 60,000 lb.

Step No. 4-If design for capacity operations is based upon a wheel load of 50,000 lb (single tire), the design for limited operations or emergency landings will in this case be based upon

a wheel load of $\left(\frac{50,000}{1.15}\right) = 43,500$ lb.

That is, for purposes of design a wheel load of 43,500 lb at 0.5-in. deflection for one loading is equivalent to a wheel load of 50,000 lb at 0.5-in. deflection for 10 repetitions, when the contact area remains constant.

Step No. 5-For a subgrade cone bearing value of 260 psi and a wheel load of 43,500 lb, Fig. 97 indicates that 19 in. of granular base are required for runways. For a Housel penetrometer rating of 15 blows, 21 in. of granular base are indicated. Averaging these two requirements gives a thickness of 20 in

Therefore, a runway constructed on this subgrade will require 20 in. of granular base or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 60,000 lb.

Step No. 6-The thickness of granular base required for a taxiway, apron, or turnaround, can also be obtained from Fig. 97 by interpolation. For a wheel load of 50,000 lb, a subgrade cone bearing value of 260 psi and a Housel penetrometer rating of 15 blows 34 and 36 in. of granular base are indicated respectively, giving 35 in as the average value.

Consequently, a taxiway, apron, or turnaround constructed on this subgrade will require 35 in. of granular base or its equivalent, for capacity operations by aircraft with wheel loads of 60,000 lb.

Step No. 7- It was indicated in Step No. 4 that for purposes of design, a wheel load of 43,500 lb at 0.5-in. deflection for one loading is equivalent to a wheel load of 50,000 lb at 0.5-in deflection for 10 repetitions, when the contact area remains the same

Step No. 8-The thickness of granular base required for a taxiway, apron, or turnaround, can be obtained from Fig 97 For a wheel load of 43,500 lb, a subgrade cone bearing value of 260 psi and a Housel penetrometer rating of 15 blows, 30 and 32 in. of granular base are indicated respectively, giving 31 in. as the average value.

Therefore, a taxiway, apron, or turnaround constructed on this subgrade will require 31 in. of granular base, or its equivalent, for emergency landings or limited operations by aircraft with wheel loads of 60,000 lb.

NOTE: If desired, the subgrade cone bearing and Housel penetrometer ratings can be converted to their corresponding plate bearing values, 30-in. plate, 0.5-in. deflection, 10 repetitions of load, by means of Fig. 48 and 51, respectively The required thickness of granular base for a runway, or taxiway, etc., can then be obtained by means of the steps outlined under *Sample Calculation No. 3*. It is recommended that this be done in any case to provide a check.

The second part of the note at the end of *Sample Calculation No. 2* is equally applicable to *Sample Calculation No. 4*.

HIGHWAYS

EVALUATING THE BEARING CAPACITY OF A FLEXIBLE PAVEMENT

Sample Calculation No. 5

(a) The following load test data have

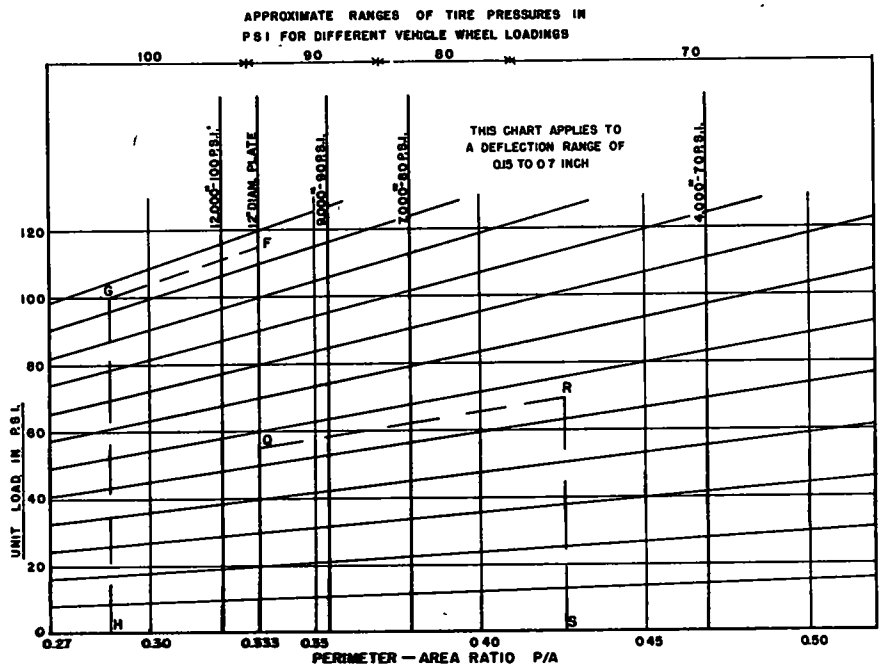

Fig. E Diagram for Evaluating Load Carrying Capacity of Highways with Flexible Pavements

been obtained at eight representative locations on the surface of a section of paved highway at the most critical period of the year, for a 12-in. plate, at 0.5-in. deflection, and 10 repetitions of load.

What is the maximum wheel load that can be carried by this section of highway for a high density of traffic?

Total Load in Pounds
6790
6710
6680
6470
6390
6340
Lower quartile point --------- 6220
6100
5980

Step No. 1–The lower quartile point is 6220 lb. Since the area of a 12-in. diameter bearing plate is 113 sq in., this corresponds to a unit load of 55 psi.

Step No. 2–The $\frac{P}{A}$ ratio for a 12-in. bearing plate is 0.333.

In general, as highway wheel loads increase, tire pressure and contact areas increase, and the $\frac{P}{A}$ ratios of the contact areas therefore decrease. Figure E is a graph of unit load in psi versus $\frac{P}{A}$ ratio. Across the top of the graph, the tire pressures which are normal for each range of $\frac{P}{A}$ ratio are given.

The data across the top of Fig. E indicate that a unit pressure of 55 psi and a $\frac{P}{A}$ ratio of 0.333 (12-in. plate) do not correspond. The problem, therefore, is to find the smaller contact area (larger $\frac{P}{A}$ ratio) and the correspondingly higher unit pressure, which are usually associated with each other.

Point *Q* in Fig. E represents a unit load of 55 psi for a 12-in. plate, ($\frac{P}{A}$ ratio 0.333). The dashed line *QR* is drawn to the right approximately parallel to the two nearest oblique lines. Its actual slope, as well as that of all the oblique lines on the graph, is such that the ratio of the unit load supported on a 12-in. plate versus that supported on a 30-in. plate is 2.4 (see Figs. 39 and A). Line *QR*

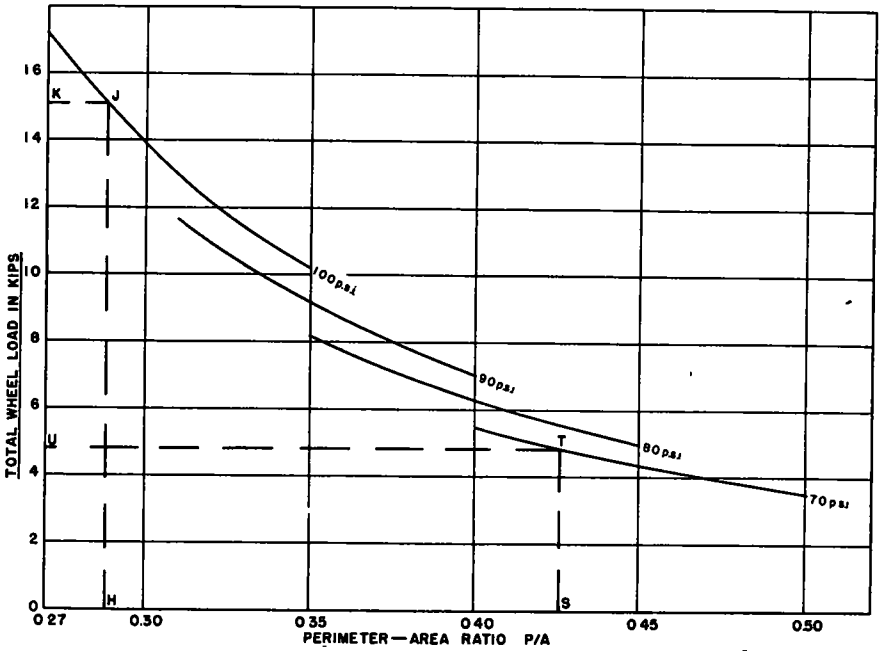

Fig. F Diagram for Evaluating Load Carrying Capacity of Highways with Flexible Pavements

is projected to *R*, the coordinates of which indicate that the unit load on the left hand side of the graph, 70 psi, is equal to the tire pressure within one of the ranges shown across the top of the graph.

The vertical dashed line *RS* through *R* cuts the abscissa at the required $\frac{P}{A}$ ratio, 0.424.

Step No. 3-The total load corresponding to a tire pressure of 70 psi and a $\frac{P}{A}$ ratio of 0.424 can be calculated, but may be read directly from Fig. F

From point *S* at a $\frac{P}{A}$ ratio of 0.424 in Fig F, the dashed vertical line *ST* is drawn to intersect the tire pressure curve labelled 70 psi, at *T*. From *T*, the horizontal broken line *TU* is drawn to cut the ordinate axis at *U*, the required value for the total load, 4,900 lb.

Therefore, the maximum wheel load which this section of paved highway can carry at a high density of traffic, is 4,900 lb

(b) The following load test data have been obtained at eight representative locations on the surface of a section of paved highway at the most critical period of the year, for a 12-in. plate, at 0.5-in. deflection and 10 repetitions of load.

What is the maximum wheel load that can be carried by this section of highway for a high density of traffic?

	Total Load in Pounds
	16200
	15600
	15400
	14300
	14100
	13600
Lower quartile point ----------	13000
	12400
	11000

Step No. 1-The lower quartile point occurs at a total load of 13,000 lb, or at a unit load of 115 psi.

Step No. 2-Point *F* in Fig. E represents a unit load of 115 psi for a 12-in. plate ($\frac{P}{A}$ ratio = 0.333). Draw *FG* to the left approximately parallel to the two nearest oblique lines, until the coordinates of *G* indicate that the value of unit pressure along the left

hand side of the graph, 100 psi, is equal to one of the tire pressure ranges shown across the top of the graph.

The vertical line *GH* cuts the abscissa at the required value of the $\frac{P}{A}$ ratio, 0.288.

Step No. 3-The total load corresponding to a tire pressure of 100 psi and a $\frac{P}{A}$ ratio of 0.288, can be read directly from Fig. F.

From point *H* of Fig. F ($\frac{P}{A}$ = 0.288), the dashed vertical line *HJ* is drawn to intersect the tire pressure curve labelled 100 psi. From *J*, the horizontal broken line *JK* is drawn to cut the ordinate axis at *K*, the required value for the total load, 15,100 lb.

Therefore, the maximum wheel load which this section of paved highway can carry for a high intensity of traffic is 15,000 lb.

Sample Calculation No. 6

The pavement on a given section of highway consists of 2 in. of asphaltic concrete and 8 in. of granular base course. No load test equipment was available, but the following cone bearing and Housel penetrometer data were obtained at eight representative locations for the top 18 in. of the subgrade at the most critical time of year.

What is the maximum wheel load that can be carried by this section of highway for a high density of traffic?

Cone bearing data psi	*Housel penetrometer data Number of blows for a penetration of 6 in.*
295	24
270	22
240	21
230	20
215	17
210	16
Lower quartile point ------ 200	-------- 15
190	14
175	12

Step No. 1-The lower quartile values are 200 psi for the cone bearing data, and 15 blows for the Housel penetrometer tests.

Step No. 2-If the base course and surface are considered equivalent to 10 in. of granular base in load supporting value, Fig. 100 indicates that for a subgrade cone bearing value of 200 psi the load supporting value of the highway is 8,800 lb, and for the Housel penetrometer rating of 15 blows, is 9,600 lb. Averaging these two values gives a load carrying capacity of 9,200 lb.

Therefore, the maximum wheel load which this section of paved highway can carry for a high intensity of traffic is 9,200 lb.

FLEXIBLE PAVEMENT DESIGN FOR HIGHWAYS

Sample Calculation No. 7

(a) What thickness of granular base is required to carry a highway wheel load of 4,500 lb at a tire pressure of 70 psi for a high density of traffic, when plate bearing tests (12-in. plate, 0.5-in. deflection, 10 repetitions of load) on the cohesive subgrade in its most critical condition, given the following data for eight representative locations on a section of highway?

Total Load in Pounds
4870
4620
4360
4230
4090
4050
Lower quartile point ---------- 3960
3870
3620

Step No. 1-The lower quartile value occurs at a total load of 3960 lb, or at a unit load of 35 psi.

Step No. 2-For a wheel load of 4,500 lb and a tire pressure of 70 psi, the contact area is 64.3 sq in. From Fig G, the corresponding $\frac{P}{A}$ ratio is 0.442.

Step No. 3-It is required to determine the unit load for a contact area of 64.3 sq in. ($\frac{P}{A}$ ratio = 0.333), which corresponds to a unit load of 35 psi on a 12-in. plate.

In Fig. H, point *17* represents a unit load of 35 psi on a 12-in. plate, ($\frac{P}{A}$ = 0.333). The dashed vertical line

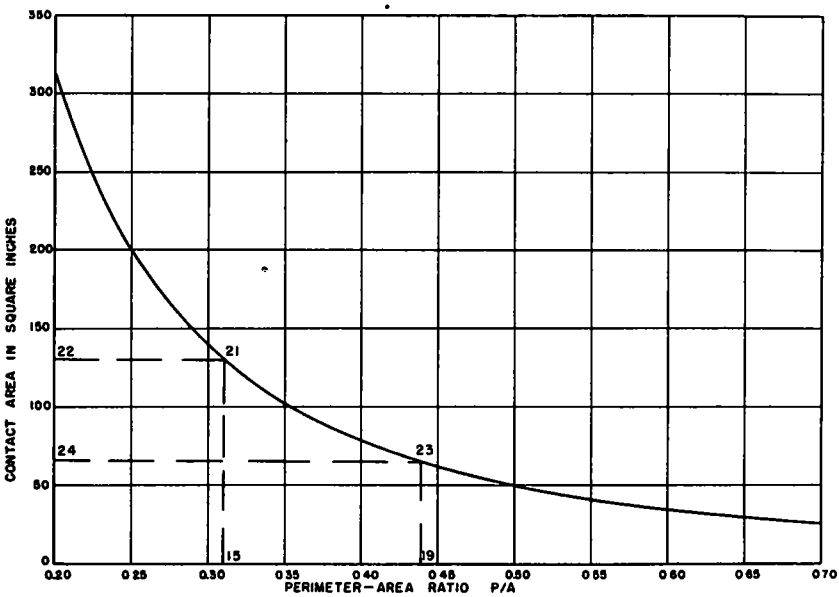

Fig. G Contact Area Versus Perimeter-Area Ratio for Highway Wheel Loadings

19,18 marks a $\frac{P}{A}$ ratio of 0.442. Broken line *17,18* is drawn to the right approximately parallel to the nearest oblique lines. The actual slope of *17,18* is such that the unit load on a 12-in. plate is twice that for a 30-in. plate (see Figs. 37 and D). From point *18*, at the intersection of dashed lines *17,18* and *19,18*, the horizontal broken line *18,20* is drawn, which cuts the ordinate axis at point *20*, at which the unit pressure is 44.5 psi. That is, the unit pressure of 44.5 psi for a $\frac{P}{A}$ ratio of 0.442 (contact area of 64.3 sq in.), is the required value of subgrade support corresponding to a unit pressure of 35 psi on a 12-in. plate. Therefore, the load supporting value of the subgrade for a contact area of 64.3 sq in. is (64.3)(44.5) equals 2860 lb.

Step No. 4-For a subgrade support of 2860 lb and an applied wheel load of 4500 lb, interpolation in Fig. 99 indicates that the required thickness of granular base is 13 in.

The necessary thickness may also be calculated directly by means of equation (15)

$$T = 65 \log \left(\frac{4,500}{2,860}\right)$$
$$= 12.7 \text{ in.}$$

Therefore, the section of highway constructed on this subgrade will require 13 in. of granular base or its equivalent, to carry a maximum wheel load of 4,500 lb for a high density of traffic.

(b) What thickness of granular base is required to carry a highway wheel load of 13,000 lb at a tire pressure of 100 psi for a high density of traffic, when plate bearing tests (12-in. plate, 0.5-in. deflection, 10 repetitions of load) on the cohesive subgrade in its most critical condition, give the following data for 12 representative locations on a section of highway?

Total Load in Pounds

9550		8740
9460		8690
9240		8620
9070		
8920	Lower quartile point ----------	8500
8830		8380
		8240
		8120

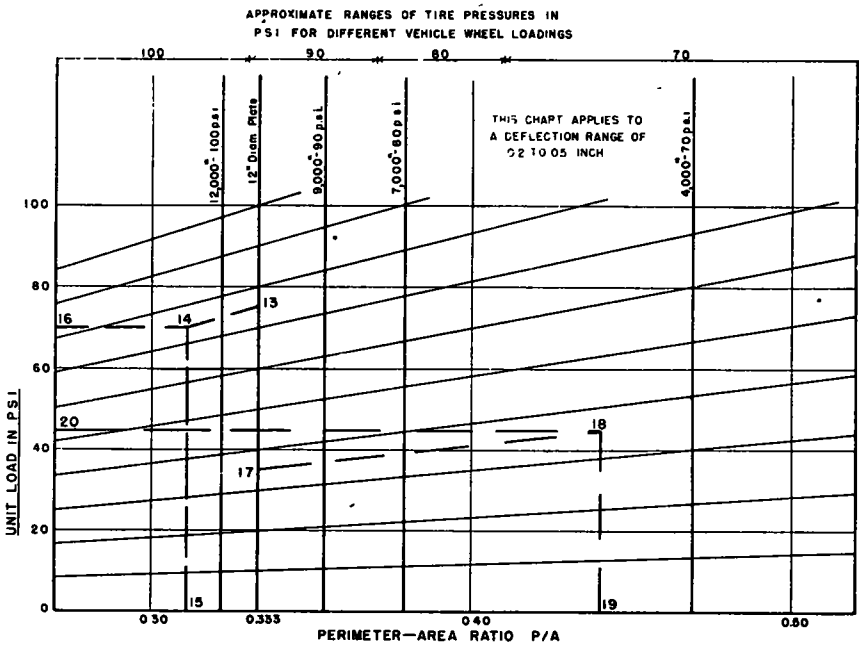

Fig. H Diagram for Evaluating Load Carrying Capacity of Subgrades for Highways (Flexible Pavements)

Step No. 1-The lower quartile value of subgrade support occurs at a total load of 8,500 lb, or at a unit load of 75 psi.

Step No. 2-For a wheel load of 13,000 lb and a tire pressure of 100 psi the contact area is 130 sq in. From Fig G, the corresponding $\frac{P}{A}$ ratio is 0.311

Step No. 3-Point *13* on Fig. H represents a unit load of 75 psi on a 12-in. plate. The broken vertical line *15,14* marks a $\frac{P}{A}$ ratio of 0.311. The dashed line *13,14* is drawn approximately parallel to the nearest oblique lines. The horizontal broken line *14,16* cuts the ordinate axis at point *16*, at which the unit pressure is 70 1 psi. That is, a unit pressure of 70.1 psi for a $\frac{P}{A}$ ratio of 0.311 (contact area 130 sq in.) is the required value of subgrade support corresponding to a unit pressure of 75 psi on a 12-in. plate.

Therefore, the load supporting value of the subgrade for a contact area of 130 sq in. is (130)(70.1) = 9,120 lb.

Step No. 4-For a subgrade support of 9120 lb and an applied wheel load of 13,000 lb, it can be determined by extrapolation in Fig. 99 that the required thickness of granular base course for this section of highway is 10 in.

The required thickness of granular base may be calculated directly by means of equation (15)

$$T = 65 \log \left(\frac{13,000}{9,120}\right)$$

$$= 10.0 \text{ in.}$$

Therefore, the section of highway constructed on this subgrade will require 10 in. of granular base or its equivalent, to carry a maximum wheel load of 13,000 lb when the intensity of the traffic is to be high.

Sample Calculation No. 8

What thickness of granular base course is required to carry a highway wheel load of 12,000 lb, when the following cone bearing and Housel penetrometer values have been obtained for the top 18 in. of the subgrade (cohesive) in its most

critical condition of moisture and density at eight representative locations?

Cone bearing data psi	*Housel penetrometer data Number of blows for penetration of 6 in.*
280	21
260	18
235	17
210	14
190	13
185	11
Lower quartile point ------ 175	--------- 10
165	9
150	8

Step No. 1-The lower quartile values are 175 psi for the cone bearing tests, and 10 blows for the Housel penetrometer data

Step No 2-For a subgrade cone bearing value of 175 psi and a highway wheel load of 12,000 lb, Fig 100 indicates that 18-in. of granular base are required For a Housel penetrometer rating of 10 blows, 21 in. of base are needed. The average for these two thickness requirements is 19.5 in.

Therefore, the section of highway constructed on this subgrade will require 19.5 in. of granular base or its equivalent to carry a wheel load of 12,000 lb for a high density of traffic.

NOTE · If preferred, the solution to *Sample Calculation No. 8* can be obtained by first converting the subgrade cone bearing and Housel penetrometer ratings to their corresponding plate bearing values, 12-in. plate, 0.5-in. deflection, and 10 repetitions of load, by means of Fig 66, or Figs 65 and 27. The required thickness of granular base for a section of highway can then be obtained by means of the steps outlined under *Sample Calculation No. 7.*

The second part of the note following *Sample Calculation No.* 2 is equally applicable to *Sample Calculations No. 6, 7,* and *8.*

DISCUSSION

Gregory P. Tchebotarioff
Princeton University

The very careful study reported by Mr. McLeod raises a number of important practical questions. Of particular interest is the reported satisfactory performance of the Dorval Airport under airplane wheel loads far exceeding the ones which follow from standard methods of design based on the CBR test. It appears essential to ascertain the causes of the discrepancy between theoretical and actual performance at that airport.

There is no doubt that the CBR test has performed an extremely important and useful function during the war period. It provided a yardstick for the satisfactory and safe design of numerous airports. The frequent criticism that such designs were too conservative could not carry much weight under wartime conditions. At the present time, however, a careful reconsideration of possibly too conservative methods is indicated. At least possible limitations of such methods should be examined. The reported performance of the Dorval airport in Canada should not be left without further study in this country.

The writer's first thought on the matter was that possibly the clay soil at Dorval might be of a type which is very sensitive to remolding. It might then be conceivable that the CBR test, being a controlled strain type of test, might produce a breakdown of the structure around the edges of the rigid plunger at an induced downward motion smaller than the one specified, with a resulting subsequent decrease of clay resistance to penetration. However, this does not appear to be a likely explanation. The writer performed a few CBR type tests on undisturbed samples of a varved clay with a comparatively high average value of the sensitivity ratio (S 12). The results did not confirm the above supposition.

Since the field CBR value in an unsoaked and undisturbed condition was also very low, a breakdown of a possibly brittle soil structure due to excessive compaction of the type reported by Mr. Bedrich Fruhauf in his very interesting paper, "Study of Lateritic Soils" also does not appear probable.

The writer therefore can only urge very strongly that the causes of the above discrepancies between design and performance be further studied, tracked down, and the results made public. Could it be that the so-called "accelerated traffic tests," which were used to correlate the CBR test data with pavement design curves, did not sufficiently emphasize the factor of distribution of traffic, the importance of which factor has been recently brought out by Mr. L. A. Palmer?[1]

Thomas B. Pringle
and Frank B. Hennion

Runways Section, Airfields Branch
War Department, Office of the
Chief of Engineers

This discussion will be limited to a brief analysis of the data obtained from Dr. McLeod's pavement investigations at 10 airfields in Canada. The paper deals with plastic subgrade, thereby eliminating two fields with sand subgrades from consideration. The data obtained from the remaining eight fields apparently form the basis for Dr. McLeod's conclusions relative to U.S. design methods

[1] *Proceedings*, Highway Research Board, Vol. 24, p. 425 (1944)

and the assumptions used in his proposed design procedure for flexible pavements.

An important consideration in U.S. design is maintenance costs. Corps of Engineers design procedures a rebased on a 25-yr service period for the pavement with nominal maintenance. Dr. McLeod's paper fails to reveal maintenance costs for the eight airfields investigated. The disregard of maintenance costs could well lead the reader to erroneous conclusions as to the economy of thin base courses in flexible pavement design. For instance, the Corps of Engineers has prepared evaluation reports for three of the 10 airfields investigated by Dr. McLeod. These fields are Ft. Nelson, Grande Prairie, and Ft. St. John. Comments of the evaluating engineer regarding reconstruction and resurfacing at these three airfields together with information for other airfields in Canada, not included in Dr. McLeod's investigation, are presented in Table A.

The data in Table A were collected during the evaluation of the airfields in 1945.

Other Canadian airfields which have failed under increased traffic are Churchill and Gander Lake. Representatives of the U.S. Army have reported visual evidence of imminent pavement failures at Uplands.

The foregoing data are presented not for the purpose of discrediting Canadian design procedures, but rather to attempt to show that light pavement designs, which prove satisfactory for the limited traffic that had been anticipated for most Canadian fields failed quickly under increased wartime traffic.

It is the opinion of the writers that the Canadian design method or procedure is substantially what has been known in this country as stage construction. Stage construction has been widely practiced in the past by highway departments particularly when funds were limited and traffic light. Flexible pavements are well suited to this type of construction because as requirements increase, thicker and higher quality pavements are added and even a certain amount of failure can be tolerated. Such information as is available to us makes us believe that whether intentional or not, stage construction is being practiced by the Canadians. Evidence of this is shown by the reconstruction and additions to the pavements listed in Table A.

Traffic: The Corps of Engineers design is based on a given loading, operating at a very high density of traffic for a period of approximately 25 yr. For heavy planes, this density of traffic is normally assumed to be 100 or more operations per day, and for light planes 1,000 to 1,500 operations per day. U.S. design presupposed that this type of operation may be continued for a 25-yr period without heavy maintenance. Dr. McLeod's paper indicates that traffic densities for the fields investigated have not approached the figures indicated above (only a very limited number of heavy planes apparently used the fields) and in addition, these traffic densities have been maintained for a period of 6 yr or less.

The 100 operations per day for heavy planes assumed for Corps of Engineers design is conservative as is shown by the following yearly traffic volumes taken from traffic reports of Army Air Force operation: Field A - 250,000; Field B - 230,000 Field C - 183,000; and Field E - 180,000. The figures quoted are the number of operations for a 1-yr period by planes in the 35,000 to 75,000 lb class. These figures do not represent traffic of the very heavy class planes, but

TABLE A

Airfield	Soil Type	Sub Base in.	Existing Base in.	Asphalt Surface Course in.	Remarks
Grande Prairie	Heavy clay	9 to 10	10	5 to 6	Runways were reconstructed in 1944-1945 and surfacing was increased from 2 in. to 5 in. minimum.
Watson Lake	Gran. in clay fines	11	8	5	1½ in. asphaltic surfacing on runways was increased to 5 in. in 1943.
Fort Nelson	Sand over clay	9 to 10	9 to 9½	5 to 6½	3 in. asphaltic surfacing on runway was increased to 6 in. + in 1944.
Ft. St. John	Plastic subgrade	9 to 13	6½ to 14	3½ to 4	Portions of runway, taxiway and apron were reconstructed in 1944.
Edmonton	Heavy clay	Concrete pavement			Asphalt surfaced taxiways and runways were reconstructed, using portland cement concrete pavement.

are representative of the heaviest class considered by Dr. McLeod's traffic data. Light plane operation for numerous fields has likewise equalled or exceeded the traffic density criteria indicated above.

In order to check design assumptions, the Corps of Engineers has maintained a record of failures occurring at Army Air Force airfields. These reports indicate in general that failures result when the pavements are subjected to intense traffic by loadings in excess of the field evaluations.

Frost Action: It is regretable that Dr. McLeod's paper does not include a discussion of the detrimental effects of frost action on flexible pavements. This has long been a problem confronting highway engineers in the northern areas of the U.S. and is accountable for a considerable number of pavement failures both on U.S. highways and airfields in areas subjected to severe winters. There should be considerable data of this nature available from investigations conducted on Canadian airfields and it is hoped that these data can be included in some future paper by the author.

In Table B there are listed several airfields in the U.S. where construction was similar to that employed by the Canadians. These airfields are representative of a much larger group at which pavement failures have occurred, and these failures for the most part can be traced to inadequate base course placed on frost susceptible subgrades.

Saturation: It is evident from Dr. McLeod's paper that an explanation of the term saturation as used by the Corps of Engineers is necessary. Others have commented on the practice of the Corps of Engineers basing their design on a saturated subgrade condition, and it appears from these comments that the difference of opinion lies in the Corps of Engineers' use of the word

TABLE B

PAVEMENT FAILURES WHERE FROST ACTION WAS CONTRIBUTING CAUSE

Data Taken from Report of Frost Investigation 1945-1946
Draft of Comprehensive Report

Airfield	Distressed Area	Subgrade Soil Type	Pavement Thickness in.	Base Thickness in.	Description of Distress of Failure
Billy Mitchell Cudahy, Wis.	2000 ft of runway system 2000ft of apron and taxiway	CL	2½ to 4	7 to 12	1942 - Runways and taxiways showed some cracking and rutting in the summer. 1943 - During spring additional cracks and ruts 2 and 3 in. deep appeared.
Strother Airfield, Winfield, Kan.	Extension areas of runways and taxiways	CL CH	1½ to 2	6 to 7½	1944-1945 - Excessive rutting and shoving of surface course, cracking and pot holes. Freezing and thawing deteriorated base.
Lewiston Airport, Lewiston, Mont.	Portions of runways and taxiways	CL	4¾ to 6	6 to 12	1943 - Pavement cracked and depressions 2 to 6 in. deep appeared. Ruts and "break through" by wheels when frost left ground.
Glasgow Airfield, Glasgow, Mont.	Large and small areas on runways	CL	5	8	1943 - Areas showed no signs of distress until frost left ground. Depressions then formed up to 6 in. in depth in surfacing Insufficient base contributing factor.

saturation. The term is used loosely by the Corps of Engineers to describe a condition of moisture found in a soil sample placed in a 6-in. mold and soaked for a 4-day period. A shorter emersion period is allowed for pervious soils. A surcharge weight equal to weight of pavement and base course is placed on the top of the sample and the sample immersed in water allowing free access of water to the top and bottom of the sample. At the end of the soaking period, free surface water is removed, and the sample allowed to drain downward for 15 min. The moisture content of samples subjected to this procedure fall generally in a range of from 75 percent to 95 percent of voids filled, depending on the soil type. This does not refer to remolded conditions under which soils can lose appreciable strength. It can be stated that the condition of saturation found by Dr. McLeod in his investigation agree in general with the findings of the Corps of Engineers.

It is difficult to reconcile the

statement of Dr. McLeod wherein he states, "Kersten, 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." It would appear that this statement is not in agreement with the preceding paragraph wherein moisture contents as high as 120 percent of the plastic limit are mentioned, and other evidence in the paper such as Figs. 3, 4, 5, and 6. If the Lethbridge airport is eliminated from the data displayed in Figs. 3, 4, 5, and 6, it would be in agreement with Kersten. There must be some reason for the low moisture content found at Lethbridge, but it is unknown to the writers and was not brought out by Dr. McLeod. Complete data on conditions at Lethbridge would be particularly interesting, especially since it is noted that a great many more samples were taken from this airfield than from any other. Complete soil data for the entire eight airfields reported would be of particular interest.

With reference to Dr. McLeod's assumptions concerning bituminous surfacing, the following is quoted:

"The test data indicate that for bituminous surfaces made with liquid asphalts, soft asphalt cements (softer than about 120 penetration), etc., 1 in. of thickness has the same load supporting capacity as about 3½ in. of granular base."

Quoting further: "For well designed and constructed bituminous concrete, penetration macadam, and sheet asphalt, 1 in. of thickness of these types appears to have the same load carrying capacity of about 2½ in. of granular base."

As a basis for the above assumptions, Dr. McLeod refers to Fig. 42 which presents his load deflection curves. The writers fail to see how the figures quoted can be arrived at from an analysis of Fig. 42 and were unable to find any other data in the report to substantiate the assumptions made. It is the opinion of the writers that the assumptions are extremely optimistic.

Also the assumption that subgrade soils might well be compacted on the wet side of optimum might lead to a dangerous practice. Corps of Engineers' experience with fine grain soils indicates that for high moisture contents, most of these soils lose their stability under the heavier compaction equipment with the resultant "bogging down" of equipment and construction delays. It has also been demonstrated that subgrade soils on the wet side of optimum lose their stability if subjected to wheel loads great enough to increase compaction.

AUTHOR'S CLOSURE

By Norman W. McLeod

The author wishes to thank Professor Tchebotarioff and Messrs. Pringle and Hennion for the instructive discussions they have contributed.

Professor Tchebotarioff's remarks were prepared on the basis of the brief summary of the paper presented at the meeting itself. His question with regard to taxiway versus runway design has been answered in the section on the design of taxiways, aprons, and turnarounds, in the text.

Messrs. Pringle and Hennion have raised a number of points on which we would like to comment.

They have inquired about the maintenance costs for the runways at the 10 airports investigated. Contrary to their expectation, apparently, the maintenance expenditures for the thin pavements on the runways at most Canadian airports

have been quite low. General runway maintenance since 1941 for Dorval and the three airports listed in Table 1, has been limited to one surface seal. In addition, occasional more or less minor patch repairs have been made, but these have been required chiefly because of blocked drainage, or of the failure to recognize and remove pockets of organic soil, or of frost-affected silt from the subgrade during original construction. The latter two items did not receive the careful attention several years ago which is given to present construction.

It was pointed out in the paper that the taxiways and aprons at most Canadian airports were originally constructed of the same design as the runways. Many of these showed signs of distress under the traffic of heavier planes, and had to be strengthened or reconstructed. This would probably not have occurred for the heavier construction now required for taxiways, aprons, and turnarounds.

It is to be emphasized that insofar as the runways themselves at Canadian airports are concerned, the district airway engineers of the Department of Transport state that maintenance costs have been very moderate.

For our part, we are inclined to question the wisdom of endeavoring to base runway design on nominal maintenance for a 25-yr period, since it is doubtful that runway requirements can be forecast that far into the future.

It is quite true, as Messrs. Pringle and Hennion point out, that stage construction has been followed to some extent in the building of airports in Canada. In view of the current state of uncertainty concerning runway design requirements for future aircraft, this procedure would seem to have considerable merit at the present time, particularly if a site is originally laid out with this in mind. From the technical articles written on the strengthening of runways in the U.S.A., it would seem that this practice is also followed to some degree by our great neighbor to the south.

In Table A, Messrs. Pringle and Hennion list five airports along the Northwest Staging Route as examples of Canadian airport construction for which the United States Corps of Engineers have evaluation data, in an "attempt to show that light pavement designs, which prove satisfactory for the limited traffic that had been anticipated for most Canadian fields failed quickly under increased wartime traffic." They also list Churchill and Gander Lake as "other Canadian airfields which have failed under increased traffic," and state that they have reported "evidence of imminent pavement failures at Uplands" airport at Ottawa.

It is to be noted first of all that Canada's Department of Transport had nothing whatever to do with airport runway construction at Churchill. The entire runway development at this airport was constructed by the Corps of Engineers, or Army Air Force, or both, of the United States War Department. Gander Lake airport is situated in Newfoundland, not in Canada, and is therefore not a Canadian airport.

The representative of the United States Army who reported "visual evidence of imminent pavement failures at Uplands" airport, must have had some other airport in mind. There is no present evidence of even incipient failure of the runways at Uplands, and our load test data indicate that it could probably carry the heaviest aircraft that can be safely operated from runways of its present length.

Mr. Homer P. Keith, District Airway Engineer for the Department of Transport at Edmonton, Alberta, has been associated with airport

construction along the Northwest Staging Route since its beginning, and is more familiar than anyone else with its construction history and its condition from time to time. Facts supplied by Mr. Keith are at considerable variance with the impression created by Messrs. Pringle's and Hennion's discussion concerning the five airports listed in their Table A.

They failed to mention that from July 1943 to March 1944, the construction of all airports along the Northwest Staging Route was taken over by the Army Air Force or Corps of Engineers, or both, of the United States War Department. Prior to July 1943, and after March 1944, the construction of these airports was the responsibility of Canada's Department of Transport. Since each took over the work of the other at various stages of completion, there is in some cases no clear demarcation between Canadian and United States design and construction.

It must also be realized that considerable urgency existed for the construction of the chain of airports, and that late in the season it was sometimes necessary, because of continuous rainy weather, to lay base course and pavement over a wet section of subgrade in order to complete the runway before winter, but with the intention of excavating and replacing these defective areas the following year, if necessary. It should be emphasized that where these areas were reconstructed by the Department of Transport, when the soft subgrade soil was removed, similar soil of a more satisfactory moisture content was employed for backfilling, and in general, the same thickness of base and pavement was used for the reconstructed areas as for the remainder of the runway or taxiway.

It is also to be noted that there are locations on the Northwest Staging Route for both runways and aprons, where heavy United States construction and lighter Department of Transport construction exist immediately adjacent to each other, and both apparently have been capable of handling all the traffic to which they were and are subjected.

The remarks of Messrs. Pringle and Hennion suggest that the strengthening and reconstruction of the runways at the airports along the Northwest Staging Route in 1944-1945 was due to the failure of existing construction under plane traffic. Information furnished by Mr. Keith is in entire disagreement with this. Both Mr. Keith and one of his principal assistants state that there is no knowledge of a plane at any time breaking through the pavement of a runway proper at any station on the route. Furthermore, they have no knowledge of plane traffic causing depressions in the runways which could be interpreted as an indication of incipient failure. Even at Edmonton, contrary to popular report, no heavy plane ever broke through the pavement on the original runways proper.

Heavy planes did break through the pavement when standing on taxiways and aprons at three of these airports, on several occasions, but it must be remembered that the tendency at that time was to construct taxiways and aprons of the same design as the runways. Because of more concentrated traffic, slowly moving planes, and other factors, the load carrying capacity of a taxiway is considerably less than that of a runway of the same design, and experience has indicated the need for much heavier construction for the former than for the latter.

It is to be emphasized that the strengthening of the runways at the airports along the Northwest Staging Route in 1944-1945 was carried out not because of any imminent or even incipient failure of the existing pavements under

traffic, but by way of preparation for the greater wheel loads of the anticipated heavier four-motored aircraft which were to be flown over this route.

No claim is made that the thin pavements for runways at Canadian airports can support wheel loads of indefinitely increased magnitude. Every runway has a load limit which cannot be exceeded without signs of distress unless it is strengthened. The principal difference of opinion between the United States Corps of Engineers and the engineers of Canada's Department of Transport concerning runway design, lies in the fact that Canadian experience indicates that an airplane wheel load of any given magnitude can be carried by a pavement and base course which have only a fraction of the thickness specified by USED design.

It is regretted if our paper left the impression that we would recommend the compaction of soils on the wet side of optimum moisture, since this was not our intention. We are in agreement that in general these soils should preferably be consolidated at a moisture content either at or slightly on the dry side of optimum, if this is at all possible. However, it has been a matter of observation at most airports tested so far in the Department of Transport's investigation, that if the density in place of a cohesive subgrade is less than the maximum (modified AASHO), the moisture content of the soil is likely to be greater than the optimum, Fig. 3. Consequently, it appears that insofar as the eventual load supporting values of the runways at most airports with cohesive subgrades are concerned, engineers should be interested in the strength or stability of the subgrade soil at its expected ultimate condition of moisture in the general vicinity of the branch of the compaction curve on the wet side of optimum, regardless of what the soil moisture condition may have been at the time the subgrade was compacted into place. The information presented in Figs. 101, 102, and 103 were prepared with this in mind.

In view of the absence of supporting data, we can appreciate the skepticism of Messrs. Pringle and Hennion with regard to the statements contained in the paper concerning the greater load supporting capacities of bituminous surfacings versus those of granular base courses of the same thickness. We agree that Fig. 42 is not a satisfactory basis for these statements, but because of the length of the paper they were not further amplified. However, representative information is tabulated in Table 13 for some of the test locations at which these data were obtained.

In column 9 of Table 13, ratios have been determined for the supporting value of the bituminous surfacing versus the calculated supporting capacity of an identical thickness of base course, assuming that both are placed on the existing base course at each test location. The load supporting capacity of the additional thickness of granular base was computed by means of equation (15) in each case, which according to the graphs of Figs. 90, 91, and 92, can be employed to calculate the bearing capacity of various thicknesses of base course.

The bituminous surface at Dorval consisted of penetration macadam with a sheet asphalt top course. The data of column 9 indicate that the average supporting value of this surfacing material was 3.56 times the supporting value of the granular base per unit of thickness. The ratio of 2.5 suggested in the paper is therefore conservative.

Similarly, for six test locations at Saskatoon and Fort St. John, which can be considered representative of pavements constructed with

TABLE 13

LOAD SUPPORTING VALUES OF BITUMINOUS SURFACES VERSUS THOSE OF GRANULAR BASE COURSES PER UNIT OF THICKNESS

Airport	Type of bituminous surface	Measured load at 0.5-in. deflection 10 repetitions of load: Surf.	B.C.	Subg.	Measured thickness: Surf.	B.C.	Calculated load support for equivalent thickness of base course[a]	Ratio - Supporting value of surface versus supporting value of base course per unit of thickness[b]
1	2	3	4	5	6	7	8	9
		lb	lb	lb	in.	in.	lb	
Dorval	Pen. mac.	29100	19000	12500	4.0	10.0	21,900	3.5
"	and	49600	27400	18700	5,0	13.0	32,700	4.2
"	sheet	39000	25000	18700	4.5	9.0	29,300	3.3
"	asphalt	43200	35600	20400	5.0	10.0	42,400	1.1
"		62500	49800	34200	4.0	10.0	57,500	1.7
"		61400	33600	27200	5.0	8.0	40,000	4.4
"		48100	24000	23300	4.0	10.5	27,600	6.7
							Average	3.56
Saskatoon	Soft	22600	19800	11600	2.5	5.0	21,700	1.5
"	asphalt	32400	28900	17800	2.0	6.5	31,000	1.7
"	cement	20300	16900	14500	3.0	6.0	18,800	1.8
Ft. St. John	Soft	76000	57000	34000	4.0	18.0	65,600	2.2
"	asphalt	32700	20300	13700	4.0	12.5	23,400	4.0
"	cement	34500	26200	15400	4.5	14.5	30,700	1.8
							Average	2.17

a Obtained by solving for P in equation (15) $T = 65 \log \frac{P}{S}$
where T = thickness of surface
S = load support on top of existing base course
P = load which would have been carried by additional base course equal in thickness to surface, if placed upon existing base course.

b Ratio of $\left(\frac{\text{column 3 minus column 4}}{\text{column 8 minus column 4}}\right)$.

liquid asphalts or soft asphalt cements, the data of column 9 demonstrate that the average supporting value of these surfaces was 2.1 times that of the same thickness of gravel base, all other conditions being equal. For 29 test locations where pavements were constructed with liquid asphalts or soft asphalt cements for which the required data are available, the average value of this ratio was 2.49. Consequently, the statement to the effect that 1 in. of this

type of bituminous surface has the supporting value of 1.5 in. of granular base is conservative, and in accordance with test results.

In column 9, two of the ratios for Dorval are lower than the value 2.5 considered acceptable. This is believed to be due to the fact that tests on surface, base course, and subgrade were made from 12 to 18 ft apart, and the subgrade support may not have been uniform at the three test locations. Similar observations were made at several test locations where the pavement contained liquid asphalt or soft asphalt cement binders.

There is one basis on which these ratios of 2.5 and 1.5 might be criticized. Air temperatures under the load test units reached a maximum of about 95° F. during the summer, and the pavement temperatures during the test were therefore often lower than when exposed directly to the sun. On the other hand, a considerable number of load tests were made on the surface during the day after the pavement had been thoroughly exposed to the sun's heat. It is possible that at maximum summer pavement temperatures, the ratio of pavement versus base course support would be lower than those suggested, although at the same time, as the data of Table 13 indicate, the ratios actually obtained were on the average considerably higher than the values of 2.5 and 1.5 given in the text.

Since the preparation of the paper, further study has been made of the subject matter presented under the heading SELECTION OF BASE COURSE MATERIALS BY THE TRIAXIAL TEST. While the method of selecting base course materials with both cohesion and angle of internal friction, which was outlined in this section, appears to be correct insofar as it goes, additional study has indicated it to be unnecessarily restrictive, and a wider range of these materials could be employed to meet a given stability requirement than it would permit. This is too comprehensive a topic to discuss further here, and it is expected to be made the subject of another paper in the near future.

Messrs. Pringle and Hennion question the statement that the field moisture contents for the cohesive subgrades at eight airports were generally less than the plastic limit. They also question the observation that this conclusion is not in agreement with that of Kersten, who found as a result of a study of available moisture data in the U.S.A., that the field moisture contents of clay subgrades usually exceeded the corresponding plastic limits.

We believe that these questions are easily answered by reference to Fig. 6, since statistically speaking 63.8 percent of all the points plotted in Fig. 6 lie below the line labelled "100 percent PL." Messrs. Pringle and Hennion suggest that if the points for Lethbridge airport are eliminated, Kersten's conclusion would apply to Fig. 6. Such, however, is not the case, for even if all the Lethbridge data were deleted, 55.1 percent of the remaining points would still lie below the line representing 100 percent of the plastic limit.

It is not correct to state that "a great many more samples were taken from this field (Lethbridge) than from any other," although because of an optical illusion, the small circles employed as symbols for Lethbridge data may make it appear that way. In Fig. 6 there are 77 points for Lethbridge, 74 points for Fort St. John, and 68 for Grande Prairie. Since the subgrade moisture information for Lethbridge is representative of a considerable area in western Canada, we see no good reason for eliminating data for this airport from Figs. 3

to 7. We believe that the subgrade moisture content for the runways at Lethbridge airport is normal for the climate, water table, and clay soil encountered in that region.

As further evidence that field moisture contents for cohesive subgrade soils in place were not greater than their corresponding plastic limits at most of the airports investigated, the ratio of field moisture content to the corresponding plastic limit were calculated for each test location at the eight airports with cohesive subgrade soils and an average ratio was then determined for each airport. These average ratios are listed in Table 14.

TABLE 14

Average ratio of subgrade moisture content in place versus corresponding plastic limit for all test locations at eight airports with cohesive subgrade soils.

Airport	Average Ratio of Subgrade Field Moisture Content to Subgrade Plastic Limit Percent
Lethbridge	65.0
Grande Prairie	85.0
Regina	85.1
Saskatoon	97.0
Fort St. John	99.3
Dorval	100.1
Winnipeg	108.0
Malton	114.3
Overall Average	94.2

The data of Table 14 demonstrate that the field moisture content may exceed the plastic limit for some airports, depending upon the prevailing local conditions. Insofar as the overall average for these eight airports is concerned, however, the field moisture content was 94.2 percent of the plastic limit. At two airports the average field moisture content exceeded the plastic limit, at three airports it was approximately equal to the plastic limit, and at three other airports, it was considerably less than the plastic limit.

The ratios contained in Table 14 are average values obtained from the actual data shown graphically in Fig. 6. Runway design, however, should probably be based upon somewhat higher ratios, (the upper quartile point might be used), and for this reason it was pointed out in the paper that it might be desirable to consider subgrade moisture contents up to 120 percent of the plastic limit, (depending upon prevailing local conditions), when designing runways in areas of wetter climate. For conditions similar to those in the Lethbridge area, on the other hand, assuming an ultimate subgrade moisture content equal to 80 percent of the plastic limit would probably provide an adequate safety factor for runway subgrade design.

The data of Table 14 indicate that the anticipated subgrade moisture content for any individual airport could be seriously overestimated or underestimated, if it were assumed upon the basis of a statistical average of data for half a continent, or for any large geographical region covering a wide range of climatic, ground water table, and soil conditions, that the equilibrium field moisture content of a cohesive subgrade soil under pavement was equal to, less than, or more than the corresponding plastic limit. Table 14 emphasizes the need for determining the ratio of field moisture content versus corresponding plastic limit for cohesive subgrade soils under pavement in the immediate general area of each proposed airport site, all other conditions being similar, if the ultimate moisture content to be expected for the subgrade under the new runways is to be estimated from

TABLE 15

AVERAGE MOISTURE CONTENT AND SATURATION DATA

Airport	Undisturbed or field condition		After soaking according to standard CBR procedure			% Sat. undist. / % Sat. after soak.	% Moist. undist. / % Moist. after soak. (whole sample)	% Moist. undist. / % Moist. top in	% Moist. top in. / % Moist. undist.	% Moist. after soak. (whole sample) / % Moist. top in.
			Whole Sample		Top in.					
	Moist.	Sat.	Moist.	Sat	Moist					
1	2	3	4	5	6	7	8	9	10	11
	%	%	%	%	%	%	%	%	%	%
Lethbridge	13.1	44.7	22.7	90.3	31.7	49.5	57.7	41.3	242	71.5
Grande Prairie	21.8	73.5	28.5	91.7	38.1	80.3	76.5	57.2	175	74.9
Regina	26.1	69.0	36.0	89.2	39.3	77.3	72.5	66.4	150	91.6
Fort St. John	19.8	84.5	22.6	92.1	29.7	91.8	87.7	66.7	150	76.1
Malton	17.9	83.3	19.7	92.4	26.4	90.2	91.0	67.8	147	74.7
Winnipeg	30.1	82.7	34.3	92.1	43.0	89.9	87.8	70.0	143	79.7
Saskatoon	25.0	78.0	29.9	91.6	35.6	85.2	83.7	70.3	142	84.0

its plastic limit with reasonable accuracy.

It is suggested by Messrs. Pringle and Hennion that the traffic for four Canadian airports is less than that upon which USED design is based. In this connection, may we again repeat a quotation from a report by the Vicksburg, Mississippi Experiment Station of the U.S. Corps of Engineers on Certain Requirements for Flexible Pavement Design for B-29 Planes. "Where failures occur in flexible pavements, they occur in a relatively few operations rather than over an extended number." As indicated previously, Canadian runways which are safe for wheel loads of only 2000lb and 5000 lb, according to U.S. Corps of Engineers' design charts, have handled tens of thousands of operations of aircraft with wheel loads of 7000 to 12,000 lb, and 25,000 to 30,000 lb, respectively. We believe therefore, that the traffic at the four airports has been much more than ample to meet the requirement of the above quotation, namely, "a relatively few operations."

Because of better compaction, and the reorientation of the soil and aggregate particles into a more stable structure, a high intensity of traffic is likely to be beneficial to runways rather than otherwise, provided always that it is reasonably well distributed over the runway surface, and that the wheel loads are within the bearing capacity of the runway structure.

We have observed from Part XII Chapter 4 of the Engineering Manual, July 1946, that the U.S. Corps of Engineers may require a greater thickness of granular base as an insulation course against frost action in the subgrade, than is indicated by the CBR rating of soaked subgrade sample. In the example worked out in this reference, for a wheel load of 60,000 lb, a thickness of 24 in. of granular base is required according to the soaked CBR

rating of the subgrade. However, because the subgrade is subjected to frost penetration in winter, the reference indicates that 46 in. of base course must be employed, and the example states that 46 in. of base course should actually be specified for design. It has been pointed out several times that in the experience of Canada's Department of Transport, the thickness of base course required for airport runways according to USED design charts based upon the CBR rating of soaked subgrade samples, are unnecessarily conservative. When it is found that these already excessive thickness requirements may be approximately doubled as a matter of routine design, merely because the subgrade exists in an area subject to frost penetration, it is to be questioned whether or not runway design is any longer associated with the economic realities which engineering organizations in most countries must consider.

The Department of Transport makes no provision for extra thickness of granular base because of frost penetration, but endeavors during construction to remove pockets of silt or fine sand occurring in the subgrade, where these are likely to develop frost leaning or frost boils. Traffic has been handled year after year right through the spring break-up period, with little or no apparent distress, as indicated by the low maintenance expenditures on the runways.

We are very much interested in the definition of the word "saturation" as employed by the U.S. Corps of Engineers, which has been given by Messrs. Pringle and Hennion. Partly on the basis of this definition, we have prepared the information summarized in Table 15, wherein averaged data for all test locations at each of seven airports with cohesive subgrades are tabulated. We regret that similar data for Dorval are not available.

The data for columns 2, 3, 4, 5, and 6, were averaged from the considerable mass of data obtained for each item for each airport from undisturbed samples sent to the laboratory for the CBR test. Percent saturation values shown in columns 3 and 5 are based upon complete filling of the soil voids with water at 100 percent.

Messrs. Pringle and Hennion state that after soaking in the standard CBR test, the voids in the soil are from 75 to 95 percent filled with water, depending upon soil type. The data of column 5 indicate that for the subgrade soils for these seven airports, the voids are at least 90 percent filled with water, on the basis of the whole sample. Column 11 demonstrates, however, that the moisture content of the whole sample is only from 70 to 90 percent of the moisture content of the top inch, and is more often only from 70 to 80 percent. It is quite probable, therefore, that the voids in the top inch on which the CBR test is made, are considerably more than 90 percent filled with water.

Column 7 contains values of the ratio of data in column 3 versus the corresponding data in column 5 expressed as percentages. That is, insofar as the values in column 7 are concerned, the data of column 5 are considered to represent 100 percent saturation, in keeping with the definition of this term employed by the Corps of Engineers. Even on this basis, the information of column 7 indicates that the cohesive subgrade soils occurring under the paved runways at these seven airports varied from only 50 to 90 percent saturation. That is, the subgrade soils at these airports were quite definitely not saturated on the average, although they had existed under the pavement from one to six years, and had in most cases probably arrived at

their equilibrium moisture content. Therefore, contrary to the statement of Messrs. Pringle and Hennion, the degree of saturation found for the subgrade in this investigation does not agree with the findings of the Corps of Engineers, if their findings are represented by the degree of saturation obtained after soaking the samples for four days.

Possibly degree of saturation is not the most satisfactory basis of comparison between field and soaked conditions. In column 8, the ratio of moisture content in the field condition versus the average moisture content of the whole sample after soaking for four days is given for each airport, that is, the ratio of corresponding data in columns 2 and 4. These ratios parallel those for degree of saturation contained in column 7, as would be expected. The data of column 9 however, tell a more important story. This column lists the ratio of moisture content in the field condition versus the moisture content in the top inch of the soaked sample on the basis of overall averages for each airport. It is apparent that the field moisture content varies from only 41 to 70 percent of that found for the top inch of the soaked samples.

It will be recalled that the CBR penetration test is made on the top of the soaked specimen, and the CBR rating of the sample is usually determined from the load supported at a penetration of 0.1 in. It is evident from the data of columns 9 and 10 therefore, that the standard CBR test is run on soil which may have a moisture content from 40 to over 140 percent greater than the field moisture acquired by the subgrade as an equilibrium condition several years after the pavement has been constructed. The strength of a moderately moist clay soil will obviously be very seriously decreased if its moisture content is increased from 40 to 140 percent.

The data of column 10 make it apparent that there is no identity and not even a similarity between the equilibrium moisture content acquired by a cohesive subgrade soil after several years under a pavement, and the moisture content in the top inch of a soaked sample of this soil on which the standard CBR penetration test is made, insofar as these seven airports are concerned.

This observation probably affords a reasonable explanation for the well-authenticated fact that the thin pavements on the runways at most Canadian airports have for years been carrying many thousands of operations of aircraft with wheel loads that are several times their safe load rating according to USED design curves. It also probably explains why no relationship could be found between the results of CBR tests made on soaked subgrade samples, and the results of subgrade plate bearing tests.

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