Highway Materials Research Laboratory
132 Graham Avenue, Lexington 29, Ky.

February 28, 1949

File: S-1-3-3
D-1-7

Memo. to: Dean D. V. Terrell
Director of Research

Re: Subgrade Bearing Beneath Flexible Pavements

At the meeting of the Research Committee last October brief mention was made of the work on bearing tests beneath flexible pavements throughout the state as opposed to the evaluation of pumping beneath rigid pavements which was completed a year ago. This project dealing with subgrade bearing beneath flexible pavements was initiated in the fall of 1947 at the request of the Division of Design. However, not more than six or eight locations were sampled and tested in that year, and field work in earnest did not start until May, 1948.

When the tests on the various highways were finished last September, 185 locations representing about 435 miles of pavement in all parts of the state had been included in the project. Considering the fact that never more than three locations were worked per day -- and this was more often two or even one location per day, not including days of travel -- just the extent of field operations represented quite an undertaking. This would have been impossible had it not been for the excellent cooperation of the District Engineers and their personnel responsible for the roads who backfilled and surfaced all the openings where tests were made after June 1.

From September to the first of December laboratory tests and calculations were in progress, and although not all of the analysis of data had been completed in those three months enough had been finished to make possible a report of the study at the 28th annual meeting of the Highway Research Board in Washington on December 10. This presentation was made at the request of the Highway Research Board Committee on Design of Flexible Pavements, of which Mr. Bray is a member. The report was very well received, there being several who discussed the paper most of whom concluded that they would like very much to have the same thing in their own states. Also, there were many requests for copies, but only a few of these were supplied.

A copy of this report entitled "An Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Flexible Pavements" is attached. Actually, the original version prepared by R. F. Baker, then Research Engineer (now Engineer of Soil Mechanics for the West Virginia State Road Commission), and W. B. Drake, Assistant Research Engineer, contained much less than the report in its present form, mainly because a number of features in the work had not been completed. In January, the written version to be printed in Highway Research Board Proceedings was prepared and it contained much more than the original oral version, these supplements consisting of thorough analyses of moisture content and density relations between soils tested in the field as opposed to the same soils tested as laboratory samples; variations in moisture contents for different seasons of the year and at different locations with respect to the pavement; factors introduced by the composition of the base material including amounts of fines and corresponding Plasticity Indexes; and other things that could be quite influential in the performance of the pavements.

Even in that enlarged Highway Research Board version, the original curves showing relationships between bearing values and indicated thickness of pavement shown in Figs. 7 to 9 and Figs. 12 and 13 were retained. These represented the greatest degree of consistency (referred to as "degree of accuracy") for the data taken at face value and having no modifications. It was known at that time that modifications based on subsequent evaluation would be desirable, because certain features of the data as well as certain conditions existing at some of the locations called for these modifications. Accordingly, some modifications have been made within the past month, and these are summarized in Table 9. With those modifications, the greatest consistency or degrees of accuracy which could be obtained with the different types of test are represented by curves in Figs. 18 to 22.

Each of these sets of curves could serve as a basis of design, however the practical aspects limit these only to tests performed in the laboratory. Therefore, the Research Laboratory is recommending the curves in Fig. 22 as "design curves", even though they have the lowest degree of accuracy of all in predicting conditions as they were actually found on the highways during this investigation. These do not represent a departure from past procedures, because the soil test procedure required is the one used for this purpose for the past several years, and the curves are based on "minimum" C.B.R. values introduced with the Pumping Study last year. They do have a radically different method of traffic calculation in the equivalent 5000 pound wheel load system as opposed to the old 22,000 pound axle load arbitrarily assumed.

This Equivalent Wheel Load method was developed in California several years ago, and inasmuch as it is rather difficult to describe and use unless one is accustomed to dealing with traffic evaluation methods, only a very brief summary of the method is contained in this report. All traffic measurements at the 27 loadometer stations represented were taken by the Division of Planning, and everything pertaining to traffic has been thoroughly discussed with Mr. Bagby and Mr. Pulliam. It is my understanding that they think predictions of traffic by this EWL system can be given with much more assurance and accuracy than would have been possible with the 7 ton axle system which we used in the Pumping study. Incidentally, if that is so, our traffic factors in the Pumping report will be revised to this system.

In view of the fact that no agreement could be obtained between field CBR values from tests beneath the pavements and laboratory CBR's from corresponding tests in the laboratory, there is good possibility that differences in moisture and density between the two are of real significance. Obviously, the better the laboratory test represents the ultimate field condition the more reliable it is for the selection of design values. This discrepancy could account for the laboratory CBR method producing the lowest degree of accuracy of any of the other methods, all of which were based on tests in the field. As a minimum an investigation of the laboratory CBR compaction and soaking methods for the possibility of better correlation is in order, and such an investigation is proposed as a supplement to this project.

The attached report is termed a semi-final report in contrast with the final report which should be ready in a few weeks. That final report will not change the substance of this report in any way unless we are able to complete our investigation of the CBR test before that time. The primary reason for the final report, however, is to record all of the vast tabulations of data which substantiate the condensed information, and to prepare a manuscript which would be suitable for publication as a Bulletin of the Engineering Experiment Station at the University. A proposal to that effect will be made at the meeting of the Research Committee on March 2.

We earnestly recommend the laboratory CBR test as it stands, the Equivalent Wheel Load method of traffic evaluation, and the curves in Fig. 22 of this report as the best basis for design of flexible pavements in Kentucky for the present. Some slight improvements over this may be possible in the near future, but until that time the Research Laboratory considers this single conclusion the most satisfactory answer to the problem outlined by the Division of Design a year and a half ago.

Respectfully submitted,

L. E. Gregg

L. E. Gregg
Associate Director of Research

LG:vk

Copies to:
Research Committee Members

Commonwealth of Kentucky
Department of Highways

Semi-Final Report
on
Project Number S-3

INVESTIGATION OF FIELD AND LABORATORY METHODS FOR EVALUATING SUBGRADE SUPPORT IN THE DESIGN OF HIGHWAY FLEXIBLE PAVEMENTS

by

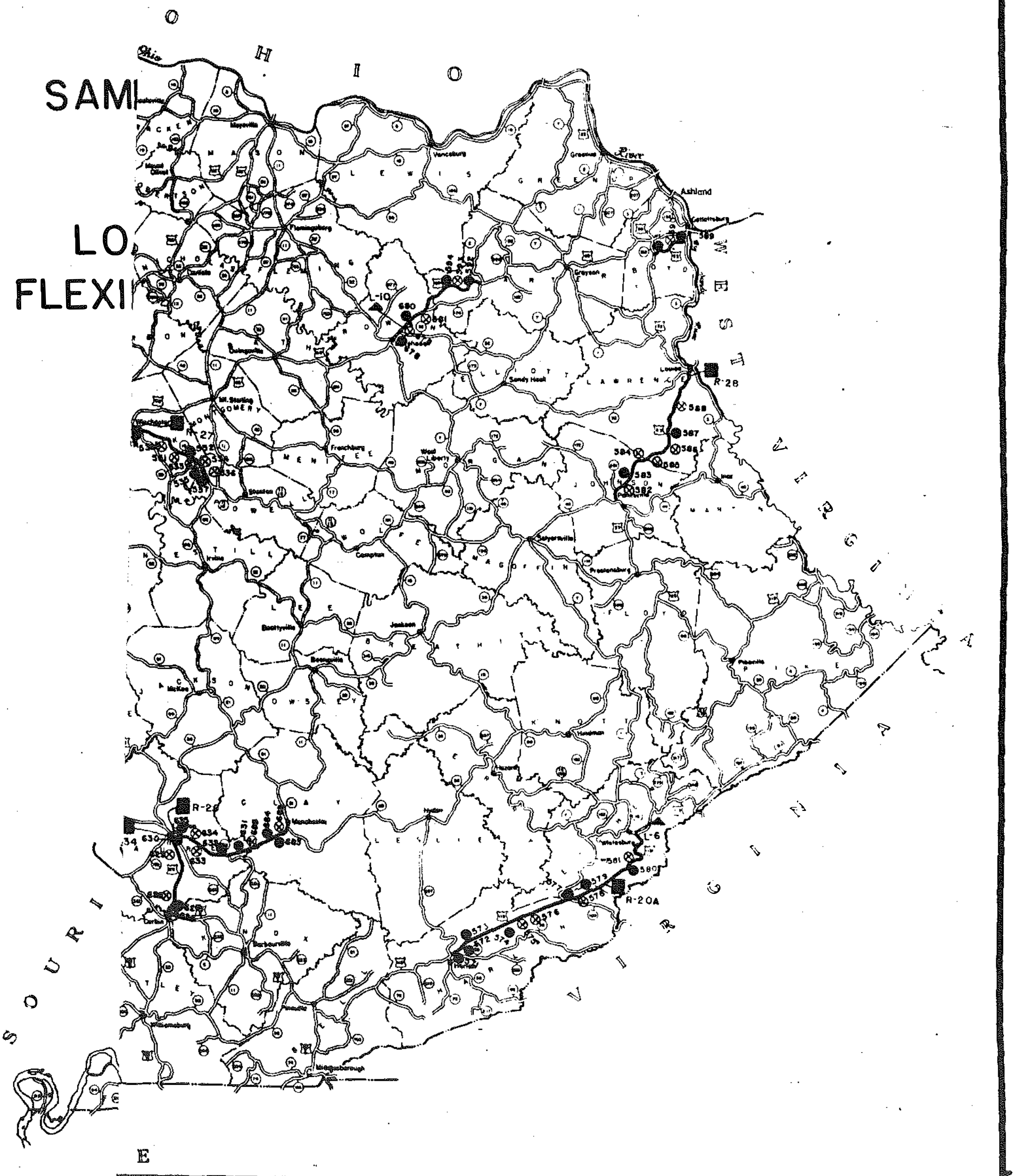
R. F. Baker, Research Engineer
W. B. Drake, Assistant Research Engineer

Highway Materials Research Laboratory
Lexington, Kentucky
March 1949

HIGHW

SAM

LO
FLEXI



LIST OF FIGURES

List of Figures (Continued)

LIST OF TABLES

INVESTIGATION OF FIELD AND LABORATORY METHODS FOR EVALUATING SUBGRADE SUPPORT IN THE DESIGN OF HIGHWAY FLEXIBLE PAVEMENTS

By

R. F. Baker, Research Engineer, and W. B. Drake
Assistant Research Engineer, Kentucky Department of Highways

ABSTRACT

Four different methods of evaluating subgrade support under flexible pavements were studied: (1) Field CBR; (2) North Dakota Cone; (3) Bearing Plates; and (4) Laboratory CBR.

Approximately 435 miles of flexible pavements in Kentucky were represented. The roads were selected so as to give a wide range in conditions of traffic, soil areas, and design. A total of 185 locations were investigated, and 338 cone tests, 291 field CBR's, and 117 series of plate tests were conducted. There were 178 subgrade samples for which the laboratory CBR test was conducted. Undisturbed samples for future triaxial tests were obtained at 21 locations.

Subgrade moisture variation was considered. Traffic was determined by loadometer surveys and use of traffic flow maps.

For the traffic imposed, adequacy of the designs -- as indicated by the presence or absence of base failures -- was evaluated from the standpoint of subgrade support measured by the four methods of test. Comparisons among the various methods of test in determining the subgrade support were made. The ultimate objective is a design criteria for flexible pavements in Kentucky.

INTRODUCTION

The basis for flexible pavement design by the Kentucky Department of Highways has been, for several years, the laboratory CBR test and the 1942 CBR curves developed by the California Department of Highways. Some modifications were applied for local conditions and observed performance. However, road performance has become so unpredictable that direct application of the empirical curves has been seriously questioned by the design engineers.

Accordingly, in the fall of 1947, the Research Laboratory was asked to evaluate for Kentucky conditions the laboratory CBR, as well as other methods currently advanced for flexible pavement design. Since such a study could very easily require several years to complete, the problem was further qualified to the extent that some recommendation was desired at the earliest possible time.

Previous work in the field of research into flexible pavement design was summarized in 1945 by the Highway Research Board Subcommittee on Flexible Pavement Design (1). The evaluation of subgrade support was, and is, the most difficult feature of pavement design. The CBR, cone penetrometer, bearing plate, and the tri-axial shear tests have been used most often for evaluating subgrade support.

Numerous highway departments including California (2), Wyoming (3), New Mexico (4), Colorado (5), and Minnesota (6) employ empirical design curves based on the CBR or modifications. A penetrometer type loading of the subgrade has been incorporated into an empirical design criterion by North Dakota (7), and into a rational design formulae by Housel (8). Kansas (9) and the Public Roads Administration (10) have applied the results of the tri-axial shear tests to a rational design criterion. Plate tests, to determine the bearing capacity of the subgrade, have been used by Campen and Smith (11) and the Bureau of Yards and Docks, U. S. Navy (12).

The approach to the problem in Kentucky was neither unique nor original. The literature concerning flexible pavement design was studied in detail, and the major problem -- the evaluation of subgrade support -- was selected as the initial objective.

SCOPE

The purpose of this investigation was to determine for Kentucky soils the effectiveness of the laboratory CBR value in designing thicknesses of flexible pavements and bases. In addition, the CBR, North Dakota Cone, and plate bearing tests were conducted in the field in order to decide which of the methods studied would give the most practical design criterion.

The study included sampling and density determination of base and sub-base materials, and field and laboratory sampling and testing of subgrade materials from 185 locations representing 434.6 miles of road selected on the basis of traffic, performance, type of subgrade soil, and types of base and surface construction. The map preceeding Page 1 shows the sample distribution. Traffic information was obtained through the cooperation of the Division of Planning. The remainder of the work was completed by the personnel of the Highway Materials Research Laboratory.

METHODS

The methods employed in the investigation could be subdivided into the following phases: preliminary, field work, laboratory testing and analysis.

Preliminary

The roads studied included those recommended by the Division of Design as being typical situations representing a variety of design.

Several others were added by the Research Laboratory so as to include every major soil area in the state.

Traffic over the selected routes was considered to be of paramount importance. Through the cooperation of the Division of Planning, 17 special loadometer stations were set up and operated in the fall of 1947. The data thus obtained, combined with those from 10 routine stations measured in early summer of 1947, furnished the basis for the analysis of traffic conditions.

Before actual field work started, detailed past design information was taken from the files. A summary of the constituents of the various projects was normally available. As would be expected, the older roads were built up through a series of projects. The main purpose of this type of information was to assist in analyzing the performance of the road. In addition, by having the information during the performance survey, it was possible to select sample locations so as to include designed variations in base and surface conditions.

Field Work

It was realized from the start that moisture conditions in the subgrade were to have a most important influence. Accordingly, in March of 1948, the first of three series of subgrade moisture samples were taken from the subgrade beneath the edge of many of the roads studied. A total of 36 such locations were sampled at that time. It was impossible to determine the location of future subgrade analysis, so only 17 were at the exact spot of subsequent subgrade testing. The second subgrade moisture sampling was completed for all locations at the time of field testing and sampling. The third measurement was made in November of 1948, at which time a portion of the former locations were visited for moisture content sampling at the edge as well as near the point of the field testing.

Base failures were the only types of pavement distress for which detailed information was obtained, since the main interest of the study was in the design of base and surface thicknesses. Classification of performance was largely limited to a visual examination of the road. Fig. 1 is a photo of a typical failure. Fig. 2 is the type of failure classed as a surface failure, and mentioned only in the performance data as a part of the evaluation of the general condition of the road.

Upon completion of the performance survey, the actual sections to be sampled were selected. At the beginning of the investigation, it was estimated that approximately one sample every two miles was the maximum density of sampling that could be completed in four months of field operations. Where the performance of the road seemed relatively uniform, and the soil areas (as judged by available geologic maps and the appearance of cuts, topography, etc.) did not change, sample locations were kept at a minimum.

The extent of field sampling and testing consisted of density and moisture content determinations for the base, sub-base and subgrade. In addition, and for the subgrade only, two CBR, two North Dakota Cone and three plate tests were conducted in as many locations as possible. Disturbed samples for laboratory analysis were taken of the base, the sub-base and the subgrade. Undisturbed subgrade samples for future tri-axial tests were obtained at 21 locations, time and soil type being limiting factors where such samples were omitted.

Many elements affected the number of tests conducted at any one location. The most significant influence was the weather. In 30 locations, tests were eliminated due to rain halting operations. Complete sampling and testing was not possible in many instances due to the variation in time required to conduct all the tests. Approximately, four men for five hours

Fig. 1. Base Failure

Fig. 2. Surface Failure

was the average required to complete one location. Greater than average depth excavation, or plastic clay subgrades that were difficult to prepare for testing often lengthened this time to six hours. With an eight man crew, three complete locations per day were the most that could be expected, and this did not include backfilling and patching by maintenance personnel responsible for the road. Thus, in order to consider a greater number of locations, if only with part of the field tests, it was decided to eliminate the plate bearing tests at approximately 25 percent, and at not more than 50 percent of the locations.

After a decision had been reached as to whether plate bearing tests would be included, the appropriate size hole was outlined on the pavement. After some experimentation and study, it was found that a 40- by 80-in. hole with plate tests, and a 40- by 40-in. opening without plate tests, were the minimum size holes that would suffice. The longitudinal edge of the hole was between one and two feet from the edge of the road. The surface was excavated with a spade bit attachment on a standard jack hammer. An air compressor mounted on a dump truck was used to drive the hammer.

In most cases, the pavement "peeled" rather readily from the base, and after it was removed, a density determination was made on the base material using calibrated sand. This latter method of density determination was used in preference to that employing a rubber balloon wherever extreme irregularities (unusual with sharp edges) existed on the surface of the material to be tested. However, the main concern over these irregularities was the difficulty in obtaining a good density determination, for it was practically impossible to prepare a level area. As a result, there can be no doubt that most of the base densities are only rough estimates of the actual density.

Moisture content samples were taken from the material removed for the base density test, and excavation of the base using the jack hammer attachment was the next step in the sampling procedure. In order to eliminate fracture of the base material sampled for future laboratory testing, the base sample was taken at a distance of at least 6 inches from the spade.

As the base was penetrated, care was taken to prevent overlooking a change in base material or an existing sub-base. If a sub-base was encountered, it was treated exactly as the base material; i.e., a density determination, moisture content sample, and a bag sample were obtained.

The hole was excavated uniformly and as each new depth was reached, observations were made for evidence of a subgrade material. When this latter material was encountered, the base or sub-base was excavated with the jack hammer to approximately one inch above the subgrade.

The final leveling was completed with small hand tools (a geologist pick, brick mason hammer, concrete trowel, and ordinary laboratory spatulas). Only small portions of the subgrade were exposed at any one time in order to minimize drying of the subgrade.

The source of reaction for the field CBR and the plate bearing tests was an I-beam welded to the under-carriage of a commercial ton and a half truck. The truck was loaded so as to give a reaction of approximately 6000 pounds. In order to eliminate considerable movement of the I-beam as the weight was transferred from the springs, the truck axle and frame were lashed together with a chain. After the excavation was complete, the truck was backed over the hole and the ends of the I-beam were jacked as shown in Fig. 3. By this method the entire 6000 pounds could be concentrated on the subgrade before there was any noticeable movement of the I-beam.

The actual transfer of the load from the I-beam to the subgrade was accomplished with a ball and socket proving ring and mechanical jack

Fig. 3. Twelve-inch Diameter Plate Test in Progress

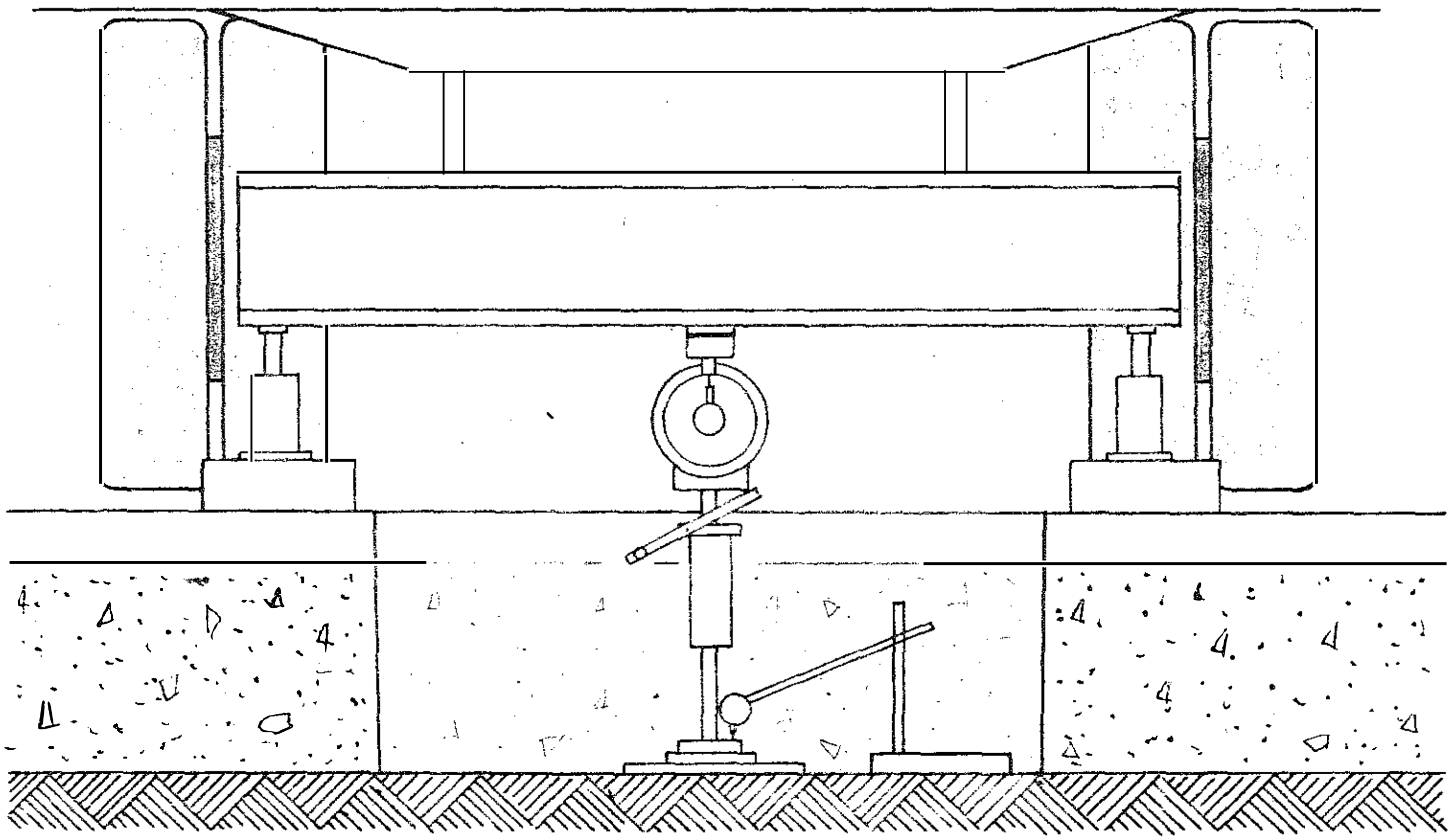
arrangement. The 10,000 pound proving ring and extensometer dial served to measure the load. For the field conditions of this study, it was not deemed necessary to have an inset in the proving ring to increase the accuracy at low ranges. The proving ring was calibrated three different times during the investigation and no appreciable change was noted.

Penetration or deflection of the subgrade was measured by a single extensometer dial as shown in Fig. 4. While it was realized that at least two and preferably three dials are recommended for plate tests (13), the additional time required for the set up made this impractical. For the relatively small plates, loads, and deflections used in this study, it was probable that the error introduced was negligible.

It can be noted in Figs. 3 and 4 that the end posts and the deflection gage standard are undesirably close to the area being tested. Unfortunately, the amount of error if any, introduced by such a situation has not been definitely determined although numerous investigators (13) have set a minimum distance of 6- to 10-feet between loaded area and dial support. Campen and Smith (14) have made some measurements for heavy loads with 12-inch and larger circular plates on base material. These indicate that for conditions in this study the maximum error in deflections that could be caused is in the range of .01- to .05-inches.

In conducting plate tests, there are four major problems of technique about which there has been considerable controversy among soils engineers; (1) size of plates, (2) rate of loading, (3) the effect of repetitional loading, and (4) the allowable deformation.

In this study, the size of the plates and the allowable deformation were limited by the reaction that could be obtained from a mobile unit that did not exceed the load limit or bridge capacities. The sizes of the plates decided upon were four, six, nine, and twelve-inch diameter circular



DIAGRAMMATIC SKETCH

of

12" DIAMETER PLATE SETUP

Fig. 4

rigid plates, and the maximum deflections were those that could be obtained for the twelve-inch plate under full load.

It was decided to load the plate, at a rate of 0.1-in. deformation per minute, in one increment up to either 0.2-in. or to the maximum load, whichever came first. The ultimate load was held until settlement was less than .003-in. per minute.

As to the repetitional loading, time permitted only three rather than the five that have been recommended by the Highway Research Board Committee on Flexible Pavement Design (13).

The procedure followed for the North Dakota Cone Test was as recommended by Boyd (7), except that the penetration of the cone was measured with an extensometer dial. Fig. 5 is a picture of the set up used for conducting the North Dakota Cone Test.

The field CBR was an "in-place" test, similar to that recommended by the U. S. Engineer Department (15). The size of the plate and the rate of loading were the same as those recommended for the laboratory CBR test.

In order to eliminate excessive excavation, a standard was established for relative positions of test on the subgrade. A sketch of this arrangement is shown in Fig. 6. A minimum spacing of one and a half diameters was required between a plate position and any other test location. Six inches was the minimum spacing permitted between positions for the field CBR, North Dakota Cone, and any other test.

Undisturbed samples were taken from 21 of the locations. While the importance of this type of sample was realized, it was necessary to eliminate some desirable details in order to get sufficient coverage of the roads. The paraffin-sealed samples were brought into the laboratory and stored in a standard "moist" room.

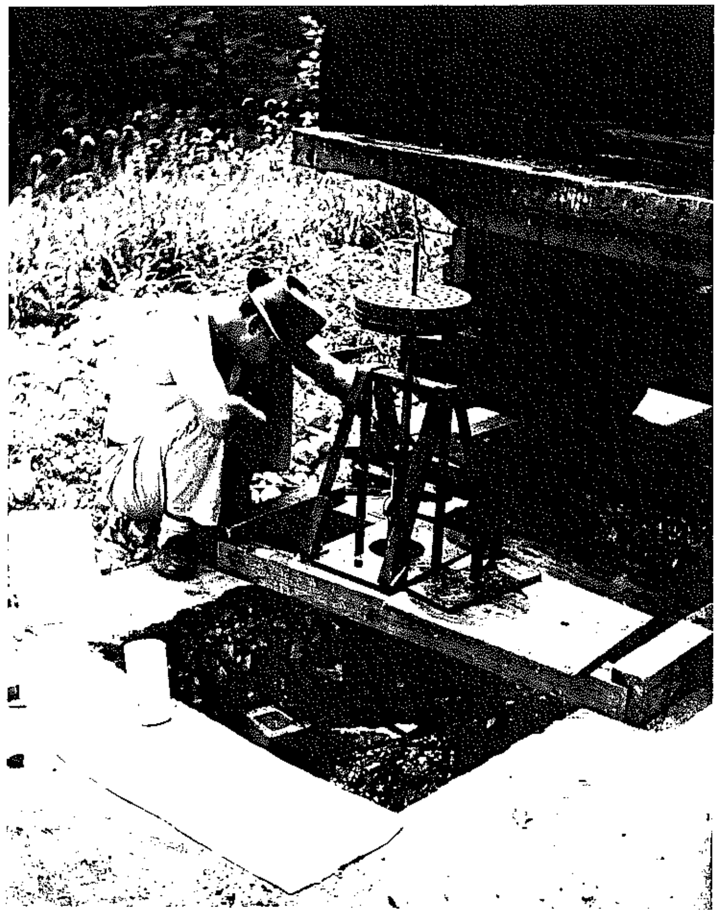
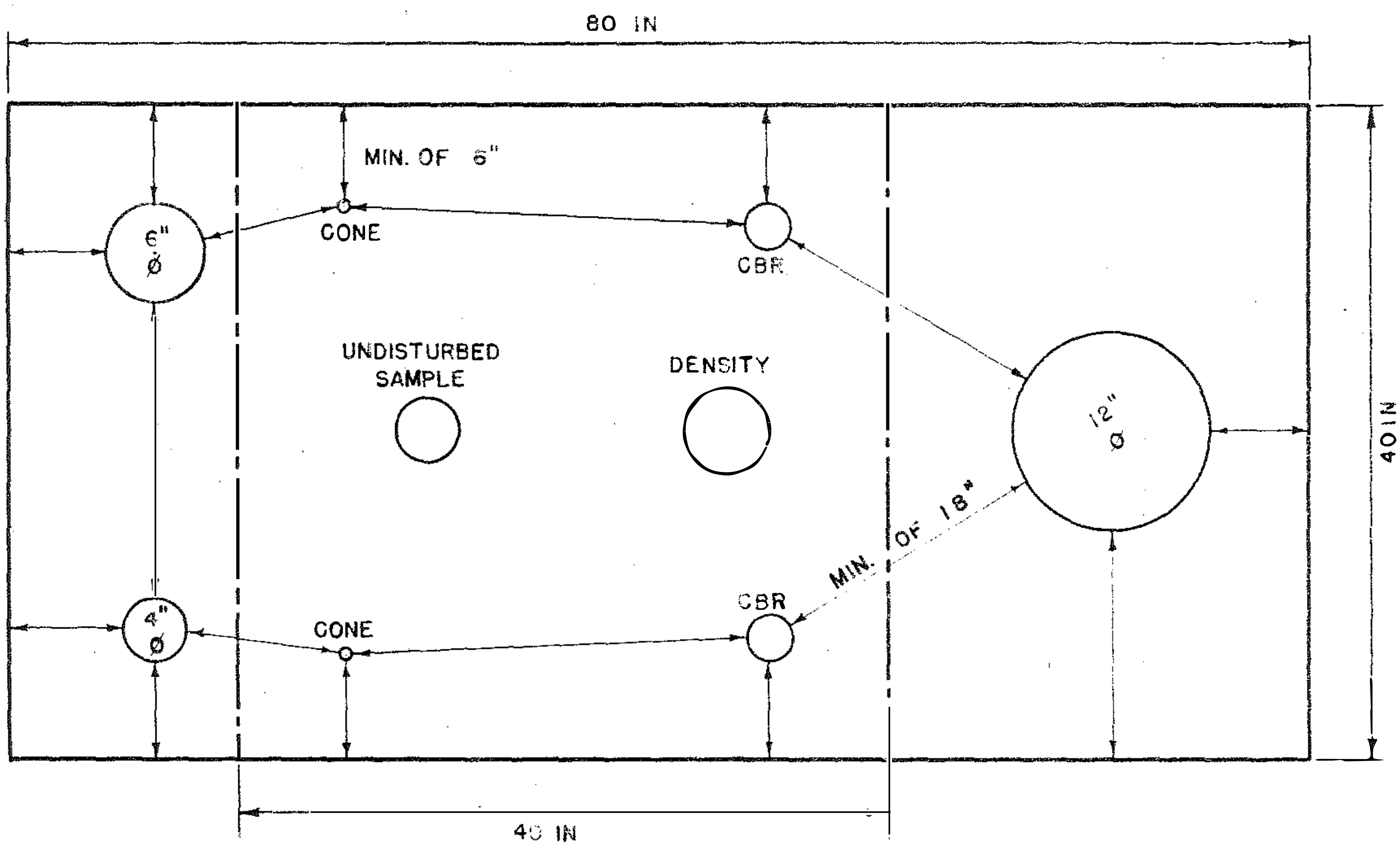


Fig. 5. North Dakota Cone Test in Progress



SKETCH OF TEST LOCATIONS IN HOLE

Fig. 6

Upon conclusion of the sampling and testing, the holes were back-filled and patched by maintenance crews responsible for the roads.

Laboratory Testing

Laboratory testing included hydrometer analyses, plasticity tests, specific gravity using a volumetric flask, standard moisture-density tests, and the CBR. All test methods were in accordance with AASHO or ASTM standards except for the CBR. The major change in the CBR from that proposed by ASTM (2) was in the soaking period for the compacted samples. Instead of the recommended four days, the samples were allowed to soak until the swell was less than .003-in. in 24 hours. This procedure originated in the Department's Testing Laboratory, due to a desire to test the soil in what was considered an extremely critical condition.

ANALYSIS

Summary of Analysis Data

Total Number of Miles Studied	434.6
Average Miles Per Location	2.34
Total Number of Locations	185
Number of Base Failure Locations	91
Number of Good Locations	94
Number of Locations with	
Laboratory CBR	158
Field CBR	138
North Dakota Cone	153
Plate Test Series	83
Undisturbed Sample	21

The most important factors influencing flexible pavement design are (1) load, (2) total thickness, and (3) subgrade support. The initial work in the analysis dealt with these three variables. The factors of lesser importance, such as quality, density, and gradation of the base; drainage; effect of cut or fill; grade; etc., were eliminated from this preliminary phase. The principle of this analysis procedure has been expressed very well by Palmer (16); "The influence of a single factor often is so outstanding that it may show a strong trend despite a high degree of variability of the other factors".

The method of approach was similar for the analysis of the four methods of evaluating subgrade support. The supporting value of the subgrade was plotted versus the total thickness above the subgrade. A traffic value for each sample was noted on the plot. The adequacy of the design at each location was indicated by a symbol representing either a base failure or a good section. Thus, the three primary factors in design were included.

By a trial and error method, curves were drawn for each traffic group so as to divide as nearly as possible the samples representing base failures from those representing good sections.

The wide range of subgrade support and mat thickness resulted in indefinite control for certain portions of some of the curves. In such cases, the curves were drawn with dashed lines so as to follow the trend of more definite parts of the curve, and to approximately parallel other better controlled curves on the same plot.

In order to compare the efficacy of the several design methods, use was made of a "percentage accuracy". These values were the ratios (expressed as a percent) of the number of locations at which the performance was correctly predicted to the total number of locations considered.

Traffic

The available traffic data included loadometer measurements from ten permanent stations operated during the period 1942-1947 and from seventeen special stations operated in 1947. In addition, total traffic on each road was estimated from flow maps for the years 1939-1947, inclusive.

The data were expanded to the total number of axles of a given magnitude (two directions) for each road and each year since the last resurfacing of the road in question. The traffic was converted into Equivalent 5000-pound Wheel Loads (referred to as EWL) by the use of the factors in Table 1. The factors are those recommended by California (17) and are adjusted from the work of Bradbury (18). The EWL value is essentially a means of including in the traffic factor the "weighted" effect of various sized wheel loads.

The total EWL values were divided into groups ranging from Group I (those with low traffic) to Group V (those with the heaviest traffic). The range in total EWL for the five groups is shown in Table 2. Also in Table 2, the "spread" of the traffic is indicated by the number of locations at which the traffic was within a given range. These figures represent all locations sampled.

The traffic calculations thus far described are estimates of past conditions. It should be pointed out that when a choice existed, estimates of past traffic were kept at a minimum. Thus, the traffic attributed to a given section would be as low as the data would permit. This procedure introduced a safety factor into a design criteria based on the past traffic values, particularly if future traffic is overestimated for design purposes.

Subgrade Moisture Content and Density

There are insufficient data to indicate conclusively the variation in the subgrade moisture content from season to season, or from edge to center of the pavement. Table 3 is a list of the moisture contents, and

TABLE 1

LIST OF FACTORS APPLIED TO WHEEL LOADS IN CALCULATING EQUIVALENT 5000-POUND WHEEL LOADS

Wheel Load	Factor[1]	Wheel Load	Factor
4500 - 5500	1	12,500 - 13,500	256
5500 - 6500	2	13,500 - 14,500	512
6500 - 7500	4	14,500 - 15,500	1024
7500 - 8500	8	15,500 - 16,500	2048
8500 - 9500	16	16,500 - 17,500	4096
9500 - 10,500	32	17,500 - 18,500	8192
10,500 - 11,500	64	18,500 - 19,500	16,384
11,500 - 12,500	128	19,500 - 20,500	32,768

[1] From "California Highways and Public Works", page 9, March, 1942.

TABLE 2

RANGE OF EQUIVALENT 5000-POUND WHEEL LOADS INCLUDED IN STUDY

Group Number	Range of Cumulative EWL (the year of last resurfacing to 1947, inclusive)	Number of Locations in Group
I	Under 1,000,000	58
II	1,000,000 to 2,000,000	60
III	2,000,000 to 3,000,000	22
IV	3,000,000 to 6,000,000	24
V	6,000,000 to 10,000,000	21
	Total	185

in Table 4 there is a summary of the data from 81 locations at which moisture measurements were made during at least two seasons of 1948.

There appeared to be some variation throughout the year, although at nearly 50 percent of the locations, the moisture contents varied by less than 2 percent. This is without regard to type of soil, depth to water table, or distance of the sample from the edge of the road. There are strong indications that for the year 1948, moisture contents were highest in the fall.

Some variation from edge to center is indicated, the moisture contents at the edge appearing to be the larger. However, since the edge samples were taken in what appears to be the more severe moisture season, this variation might well be seasonal.

In Table 5, there is a summary of the moisture content data taken at the time of the field testing, compared with the plastic limit, optimum moisture content, laboratory CBR moisture content (entire sample), percent saturation and density, and the type of soil. There are no data as to depth to water table, and the adequacy of the drainage has not been indicated.

It can be seen that at approximately 33 percent of 149 locations, the field moisture content was larger than the Plastic Limit, and at 43 percent of 161 locations the field moisture content was larger than the optimum moisture content.

For the various PRA soil groups, only the A-4, A-5, and A-5-7 groups were represented by a sufficient number of samples to estimate moisture relationships. For the A-4 soils, the field moisture content was greater than the Plastic Limit in 35 percent of the cases, and larger than the optimum in 37 percent of the cases. For A-5 and A-5-7 soils, the field moisture content was larger than the Plastic Limit in only 29 percent of

TABLE 3

LIST OF FIELD MOISTURE CONTENTS

Sample No.	Group No.	Moisture Content			Sample No.	Group No.	Moisture Content		
		Spring	Summer	Fall			Spring	Summer	Fall
505-S	B		18.0	21.7	598-S	B		12.0	17.8
510-S	B		24.0	31.4	599-S	B		17.0	16.5
512-S	B		25.0	23.8	601-S	B		33.0	25.8
513-S	B		17.0	23.0	602-S	C	10.4	14.0	14.6
514-S	B		17.0	16.3	603-S	B		17.0	15.9
516-S	B		30.0	34.0	607-S	B		32.0	16.7
518-S	B		29.0	17.8		A	20.5		20.6
523-S	B		24.0	12.0	612-S	C	19.0	23.0	20.0
524-S	B		25.0	17.2	614-S	B		11.0	19.8
527-S	B		2.9	15.9	615-S	B		14.0	16.7
540-S	C	12.1	14.0	14.0	618-S	B		19.1	20.3
542-S	B		14.0	19.0	619-S	B		8.0	11.8
543-S	C	15.5	16.0	17.1		A	13.8		14.4
544-S	C	7.3	15.0	19.6		A	12.1		13.0
545-S	C	14.3	20.0	18.1	626-S	C	9.3	12.0	8.9
547-S	C	15.3	21.0	15.7	628-S	B		12.0	11.1
548-S	C	16.6	20.0	15.8	629-S	B		11.0	12.4
	A	16.3		14.3	630-S	B		11.0	16.9
552-S	B		16.0	15.8		A	18.3		20.2
555-S	C	15.0	13.0	25.9	633-S	B		29.0	18.9
	A	4.8		20.3	634-S	C	16.5	8.0	16.5
557-S	B		16.0	15.3	635-S	C	10.0	12.0	11.4
558-S	B		14.0	15.9	636-S	B		15.0	21.7
561-S	B		19.0	14.5	637-S	B		19.0	21.3
565-S	B		17.0	19.8	642-S	B		18.0	17.5
566-S	B		14.0	16.9	643-S	B		21.0	16.9
569-S	B		19.0	21.9	648-S	B		13.0	22.2
571-S	B		14.0	18.7	649-S	B		14.0	21.2
573-S	B		18.0	15.6	650-S	B		11.0	20.0
577-S	B		15.0	18.7	651-S	B		19.0	15.7
578-S	B		10.0	10.1	652-S	B		17.0	14.3
579-S	B		11.0	14.7	656-S	B		22.0	22.4
	A	10.0		6.2	657-S	B		25.0	24.7
	A	10.1		9.1	660-S	B		11.0	18.1
587-S	B		15.0	11.6	661-S	B		8.0	17.0
	A	11.4		11.9	663-S	B		6.0	24.3
589-S	B		16.0	6.4	664-S	B		25.0	24.5
590-S	C	4.8	12.0	12.4	668-S	B		20.0	18.3
595-S	B		23.0	12.3	669-S	B		19.0	16.1
596-S	B		19.0	19.5	679-S	C	8.8	10.0	9.7
					685-S	C	15.7	14.2	16.9

TABLE 4

SUMMARY OF SUBGRADE MOISTURE CONTENT DATA
OBTAINED IN THE SPRING, SUMMER AND FALL OF 1948

Group	No. of Locations	Description	Pavement Edge March, 1948	Four Feet from Pavement Edge Summer, 1948	Pavement Edge November, 1948
A	9	No. of Locations when Moisture Content was the Larger	3		6
		No. of Locations at which Moisture Content varied by Less than 2%	6		6
B	57	No. of Locations when Moisture Content was the Larger		26	31
		No. of Locations at which Moisture Content varied by Less than 2%		17	17
C	15	No. of Locations when Moisture Content was the Larger	0	7	6
		No. of Locations at which Moisture Content varied by Less than 2% from the Average	9	8	11

TABLE 5

SUMMARY OF RELATIONSHIP BETWEEN FIELD
MOISTURE CONTENT, PLASTIC LIMIT, OPTIMUM
MOISTURE CONTENT AND LABORATORY CBR MOISTURE CONTENT

PRA Classification	Description	Plastic Limit		Optimum Moisture Content		Laboratory CBR Moisture Content (Entire Sample)	
		No. of Locations	% of Total Samples	No. of Locations	% of Total Samples	No. of Locations	% of Total Samples
A-1	Total number of Locations	0	-	2	-	1	-
	No. of Locations where the Field M.C. was = to or greater than	0	0	0	0	1	100
	Field M.C. plus or minus 2%	0	0	0	0	0	0
A-2	Total number of Locations	5	-	10	-	8	-
	No. of Locations where the Field M.C. was = to or greater than	0	0	4	40	4	50
	Field M.C. plus or minus 2%	1	20	4	40	2	25
A-2-4	Total number of Locations	2	-	2	-	3	-
	No. of Locations where the Field M.C. was = to or greater than	0	0	1	50	2	67
	Field M.C. plus or minus 2%	1	50	1	50	1	33
A-4	Total number of Locations	71	-	73	-	69	-
	No. of Locations where the Field M.C. was = to or greater than	25	35	27	37	34	49
	Field M.C. plus or minus 2%	18	25	36	49	31	45
A-4-5	Total number of Locations	0	-	1	-	1	-
	No. of Locations where the Field M.C. was = to or greater than	0	0	0	0	1	100
	Field M.C. plus or minus 2%	0	0	1	100	1	100
A-4-6	Total number of Locations	5	-	7	-	7	-
	No. of Locations where the Field M.C. was = to or greater than	2	40	3	43	4	57
	Field M.C. plus or minus 2%	0	0	2	29	4	57
A-5	Total number of Locations	52	-	45	-	39	-
	No. of Locations where the Field M.C. was = to or greater than	15	29	23	51	18	46
	Field M.C. plus or minus 2%	14	27	21	47	18	46

TABLE 5 (Continued)

PRA Classification	Description	Plastic Limit: No. of Locations	Plastic Limit: % of Total Samples	Optimum Moisture Content: No. of Locations	Optimum Moisture Content: % of Total Samples	Laboratory CBR Moisture Content (Entire Sample): No. of Locations	Laboratory CBR Moisture Content (Entire Sample): % of Total Samples
A-5-6	Total number of Locations	0	-	0	-	1	-
	No. of Locations where the Field M.C. was = to or greater than	0	0	0	0	1	100
	Field M.C. plus or minus 2%	0	0	0	0	0	0
A-5-7	Total number of Locations	12	-	14	-	14	-
	No. of Locations where the Field M.C. was = to or greater than	6	50	9	64	6	43
	Field M.C. plus or minus 2%	5	42	4	29	3	21
A-6	Total number of Locations	1	-	3	-	3	-
	No. of Locations where the Field M.C. was = to or greater than	1	100	2	67	2	67
	Field M.C. plus or minus 2%	0	0	1	33	0	0
A-7	Total number of Locations	1	-	1	-	1	-
	No. of Locations where the Field M.C. was = to or greater than	1	0	1	100	1	100
	Field M.C. plus or minus 2%	1	100	0	0	0	0
Total	Total number of Locations	149	-	158	-	147	-
	No. of Locations where the Field M.C. was = to or greater than	49	33	70	44	74	50
	Field M.C. plus or minus 2%	40	27	70	44	60	41

the situations, and larger than the optimum moisture content in 51 percent of the cases.

The data indicate that the moisture content of the field and laboratory CBR test samples were reasonably close. However, the tabulation below shows that the densities obtained in the Laboratory CBR test were considerably greater than those for the field. The same was true for the percent saturation.

SUMMARY OF RELATIONSHIP BETWEEN THE LABORATORY CBR DENSITY[1] AND PERCENT SATURATION[2] VERSUS THE FIELD DENSITY AND PERCENT SATURATION FOR 128 LOCATIONS

Description	Percent
Field percent maximum density greater than 100	48
Laboratory CBR percent maximum density greater than 100	95
Field percent maximum density greater than laboratory CBR	17
Field percent maximum density plus or minus 3 percent of laboratory CBR	59
Field percent saturation equal to or greater than 90	34
Laboratory CBR percent saturation equal to or greater than 90	87
Field percent saturation greater than laboratory CBR	10
Field percent saturation plus or minus 3 percent of laboratory CBR	20

While the field moisture content was larger than the laboratory CBR moisture content in 50 percent of 147 cases (Table 5), the field percent saturation was larger than the laboratory CBR percent saturation in only

[1]Ratio of unit dry weight of a soil to Standard Proctor Maximum Density.

[2]Ratio of volume of water to volume of voids (where voids is all space not occupied by soil particles).

10 percent of 128 cases.

Laboratory CBR

The method used for compaction and soaking of the soil resulted in a denser sample (at about the same moisture content) but a higher degree of saturation than exists in the field. The high densities obtained by the California method of compaction have been recognized by others (15).

The laboratory CBR value for each increment of penetration, as well as for the average, minimum, and maximum, were analyzed to determine which value gave the best correlation. The percentages accuracy listed in Table 6 indicate very little difference as to which penetration was used. Those developed for the minimum laboratory CBR are shown in Fig. 7, and have a percentage accuracy of 76.

After the curves had been developed, the California A and B curves (2) were plotted. The shape of the curves checked closely for CBR values up to ten. However, the traffic values were lower for the data from this study. Above the value of ten, there is a much greater reduction in mat thickness requirements for the California curves. The four- to six-inch thicknesses required by the data of this study are due to failed locations with relatively high CBR values and mat thicknesses. A complete analysis of these soils has not been made. It is possible that some belong in the category recognized by California as being particularly troublesome due to irregular grain size distribution.

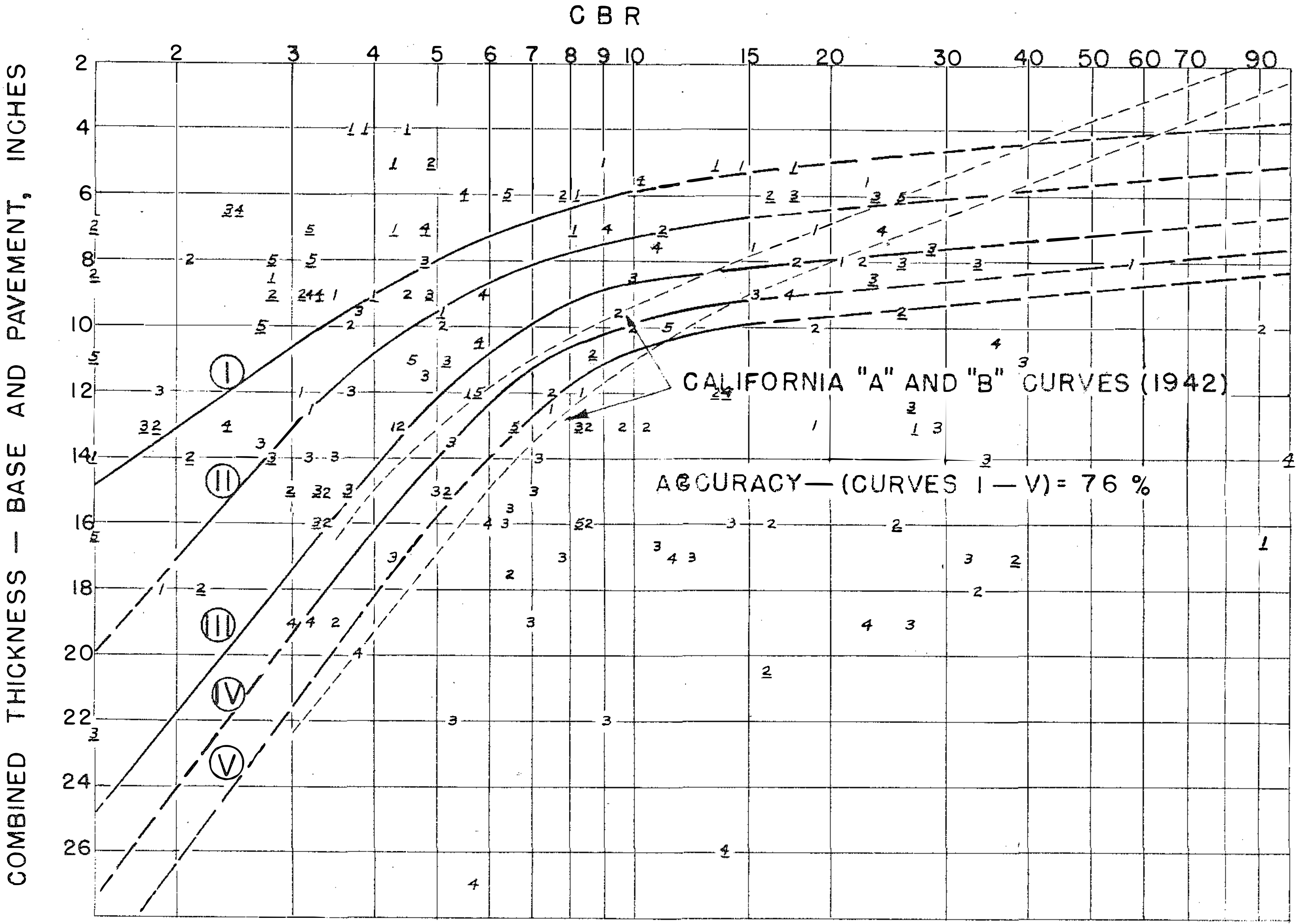
Field CBR

In Fig. 8, there is a series of curves for the minimum field CBR value. As was done with the laboratory data, the field CBR value for all increments of penetration, average, minimum and maximum were analyzed, and the results were as indicated in Table 6. The accuracy of these curves

TABLE 6

SUMMARY OF PERCENTAGES ACCURACY

Test Procedure	Test Result Considered	Traffic Groups I	II	III	IV	V	Over All Percentage Accuracy
Laboratory CBR	Average	78	69	67	74	88	74
	Maximum CBR	80	69	84	74	88	77
	Minimum CBR	87	76	74	70	83	76
	0.1	78	66	74	78	88	74
	0.2	84	68	68	74	94	76
	0.3	78	61	74	83	88	74
	0.4	84	75	61	61	83	75
	0.5	82	74	63	61	76	74
Field CBR	Average	79	71	88	81	89	79
	Maximum	82	80	88	86	95	83
	Minimum	78	71	92	84	95	83
	0.1	75	74	83	88	93	80
	0.2	75	74	91	83	86	80
	0.3	84	78	88	81	95	83
	0.4	76	75	88	81	95	80
	0.5	78	76	89	81	89	79
North Dakota Cone	Average	78	72	82	85	100	79
	Maximum	77	69	82	85	94	79
	Minimum	77	72	82	89	94	78
Plate Bearing Test	12"	74	67	87	82	100	78
	30"	71	70	87	73	86	76



C B R
COMBINED THICKNESS — BASE AND PAVEMENT, INCHES
CALIFORNIA "A" AND "B" CURVES (1942)
ACCURACY — (CURVES I — V) = 76 %
I
II
III
IV
V

Fig. 7

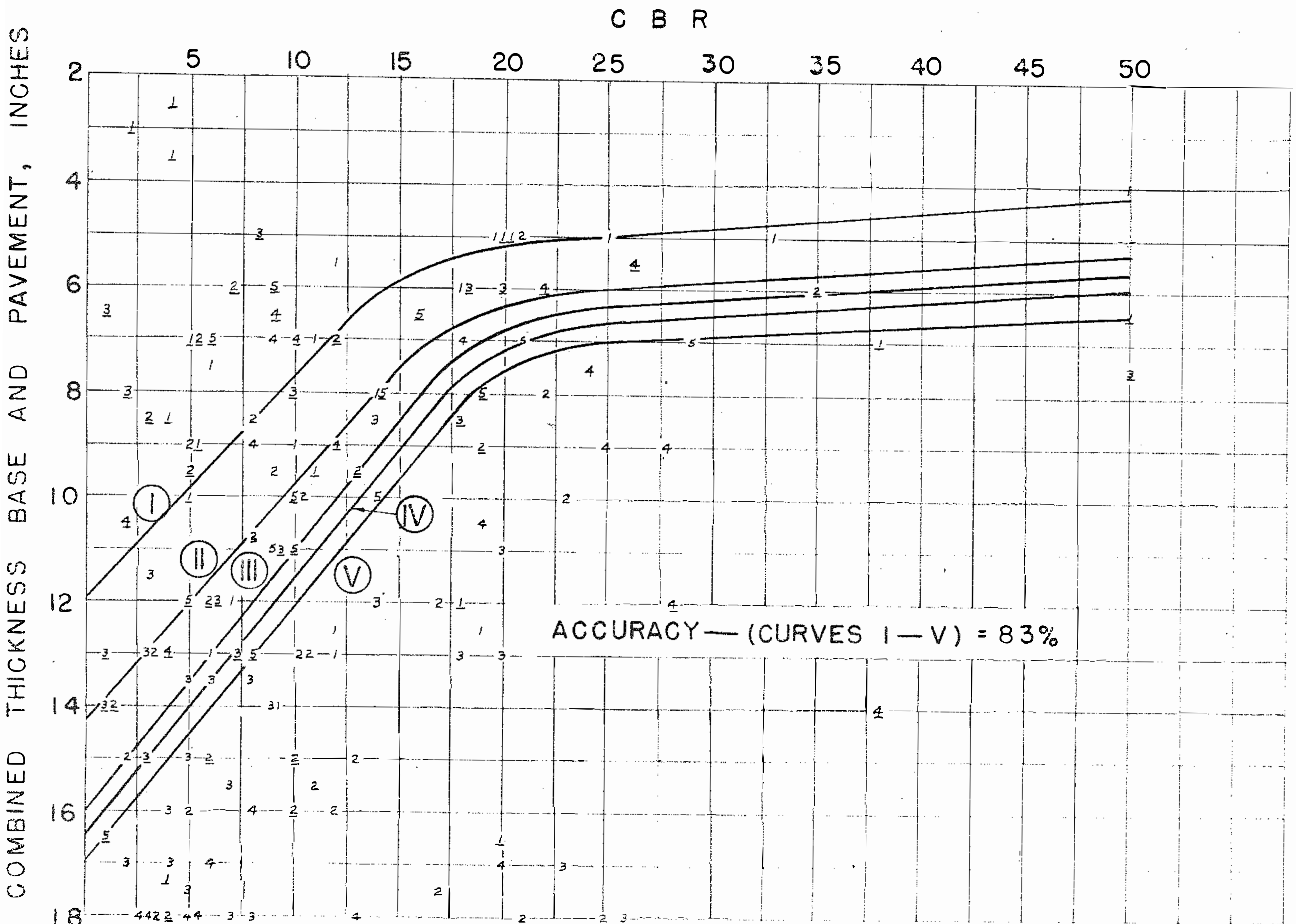

C B R
COMBINED THICKNESS BASE AND PAVEMENT, INCHES
ACCURACY — (CURVES I — V) = 83%
I
II
III
IV
V

Fig. 8

was 83 percent, 7 percent higher than for the laboratory CBR data

North Dakota Cone

The minimum, and average North Dakota Cone bearing values for a given location were analyzed and the percentages accuracy shown in Table 6. Where three tests were conducted, any test with a result not in accord with the other two was eliminated from further consideration. In Fig. 9, the curves are shown for the average North Dakota Cone Bearing value. The percentage accuracy was 79.

The design curve for the original North Dakota Cone study was plotted after an analysis was made of the data from this study. In general, the curve resembles Curve 1 developed from data obtained in this project.

Plate Bearing Tests

Initially, the field data from plate bearing tests were plotted as total load versus deformation (Fig. 10). A deformation of 0.1-inch was the maximum that was available from the data without extensive projection of the curves. The load for each plate at 0.1-inch deformation and a single loading was converted to unit stress and this value plotted versus the perimeter-area ratio (Fig. 11).

There were 68 locations tested with at least three plates. In only 23 of the analyses did the data plot in a reasonably straight line such as shown in Fig. 11. For an additional 11 cases, the points were near enough to a straight line that a fair average could be drawn. For the remaining 34 of the locations, individual plate test results within a series of three were eliminated on the basis of:

(a) irregularities of the load-deformation curves;

(b) large discrepancies in bearing value from one test with regard to the other two tests;

(c) values causing decreasing stress with increasing perimeter-area ratios.

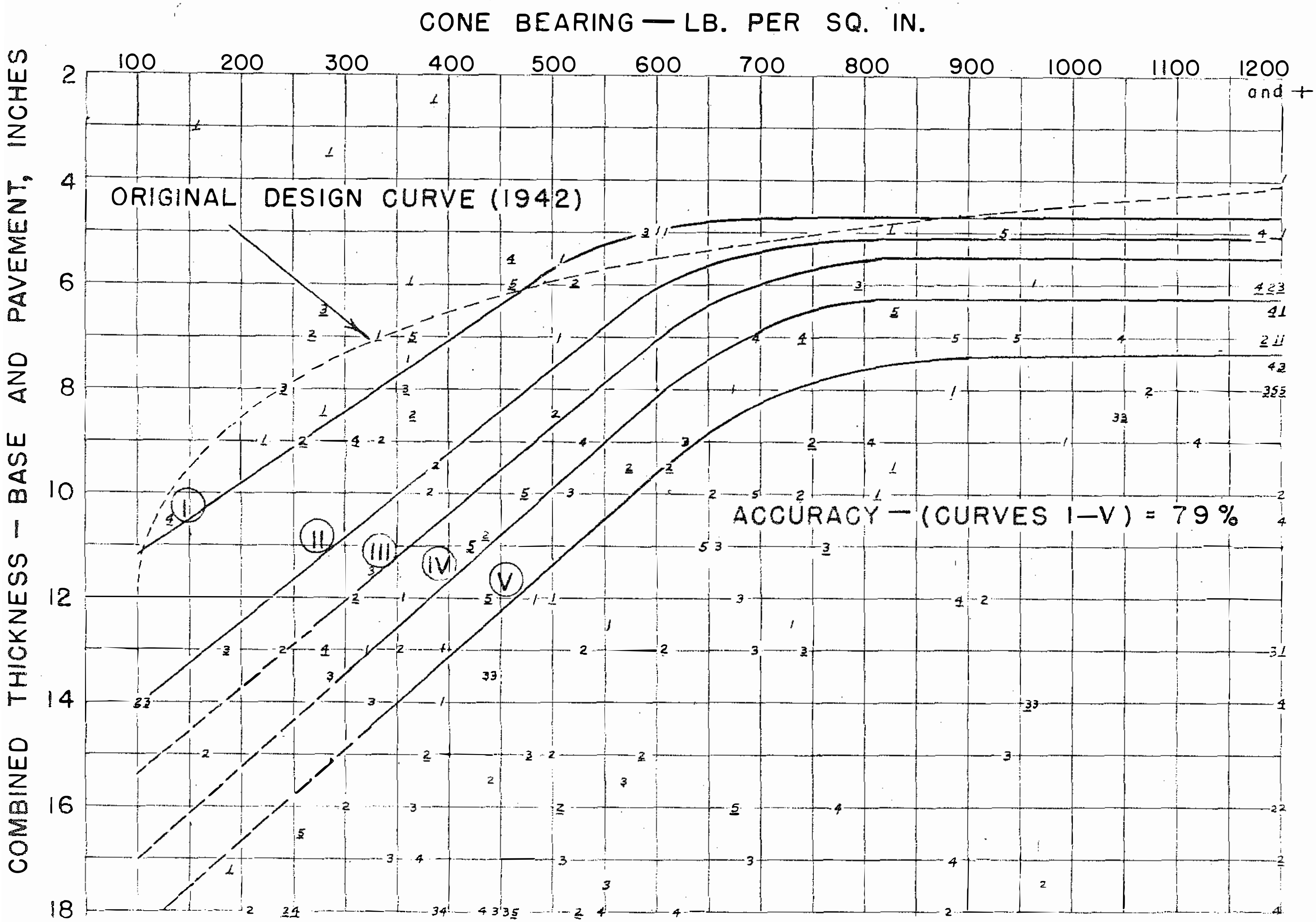
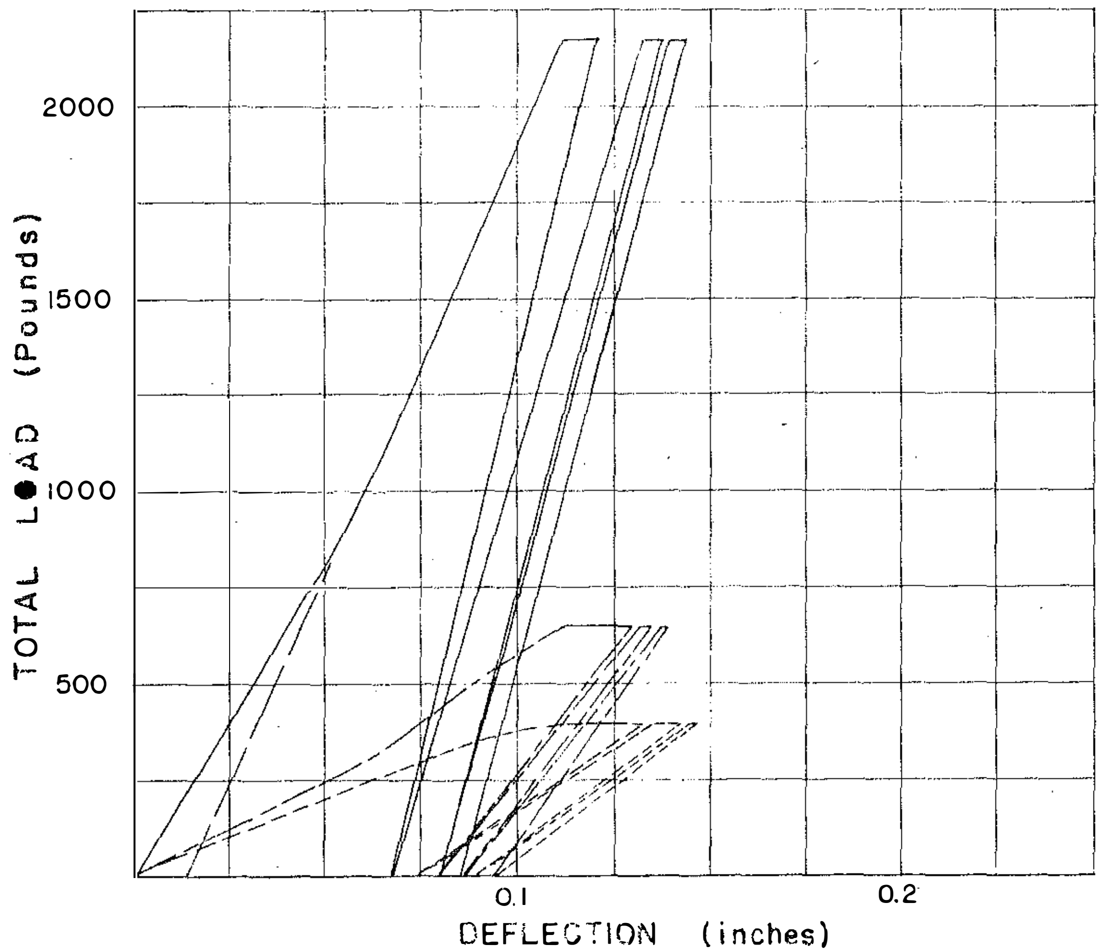
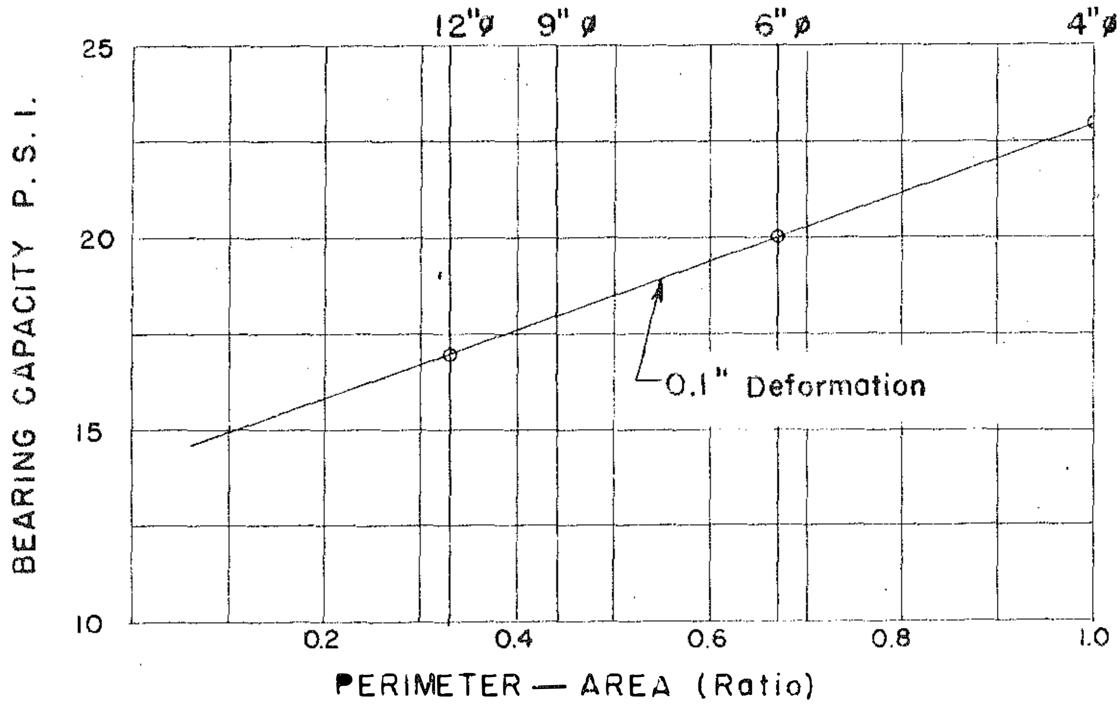

CONE BEARING — LB. PER SQ. IN.
100
200
300
400
500
600
700
800
900
1000
1100
1200
and +
COMBINED THICKNESS - BASE AND PAVEMENT, INCHES
2
4
6
8
10
12
14
16
18
ORIGINAL DESIGN CURVE (1942)
ACCURACY — (CURVES I—V) = 79%
I
II
III
IV
V

Fig. 9


TOTAL LOAD (Pounds)
2000
1500
1000
500
0.1
0.2
DEFLECTION (inches)
Fig. 10


12"ø
9"ø
6"ø
4"ø
BEARING CAPACITY P. S. I.
25
20
15
10
0.2
0.4
0.6
0.8
1.0
0.1" Deformation
PERIMETER — AREA (Ratio)
Fig. 11

Of the 34 plate tests eliminated, six were 4-inch, thirteen were 6-inch, four were 9-inch and eleven were 12-inch.

The failure of fifty percent of the samples to conform to generally accepted relationships between stress and the perimeter-area ratio could be attributed to the following factors:

(a) Working in a plate-size range of recognized questionability;

(b) irregularities in material tested;

(c) unfavorable test conditions, such as proximity of dial stand and end posts to the loaded area, and minor irregularities in rate of loading;

(d) human errors.

For the purposes of an empirical analysis, the bearing capacities plotted in Fig. 12 for 12-inch diameter plates, were taken from the plot of stress versus perimeter-area ratio rather than the actual test values. The lines were projected to a perimeter-area ratio of 0.133 to determine the bearing value under a 30-inch diameter plate.

The design curves developed from the data for the 12- and 30-inch plates show in Figs. 12 and 13, that the accuracy percentages were 78 and 76, respectively.

The effect of repetitional loading has not been analyzed to date.

<u>Base Samples</u>

The results of the mechanical analyses and plasticity tests of the base samples were considered in the light of the following recommendations made by the Highway Research Board Subcommittee on Flexible Pavement Design (19):

(1) The P.I. of the material passing the No. 40 sieve should be no greater than 6.

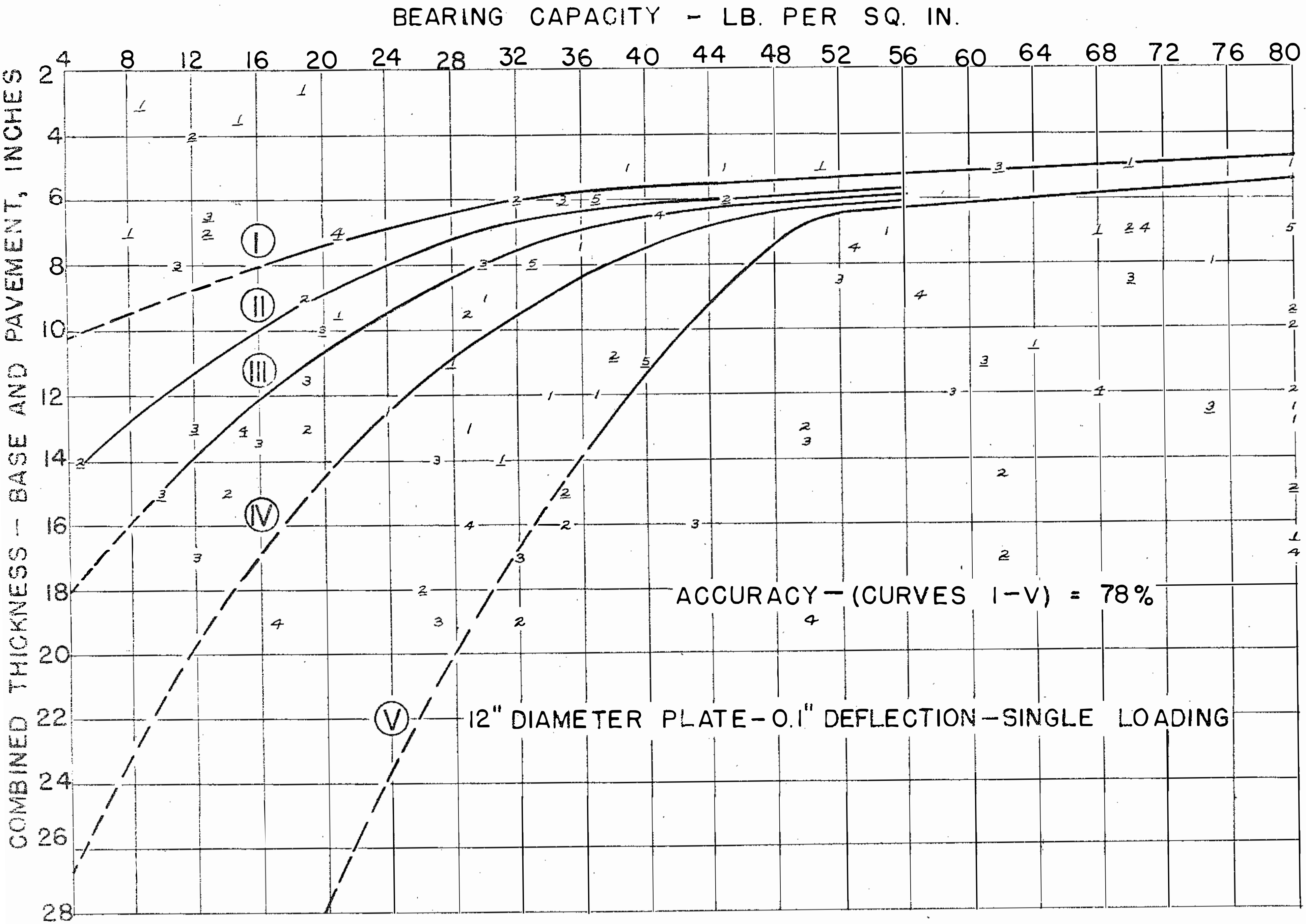

BEARING CAPACITY - LB. PER SQ. IN.
COMBINED THICKNESS - BASE AND PAVEMENT, INCHES
4 8 12 16 20 24 28 32 36 40 44 48 52 56 60 64 68 72 76 80
2 4 6 8 10 12 14 16 18 20 22 24 26 28
I
II
III
IV
V
ACCURACY — (CURVES I-V) = 78%
12" DIAMETER PLATE - 0.1" DEFLECTION - SINGLE LOADING

Fig. 12

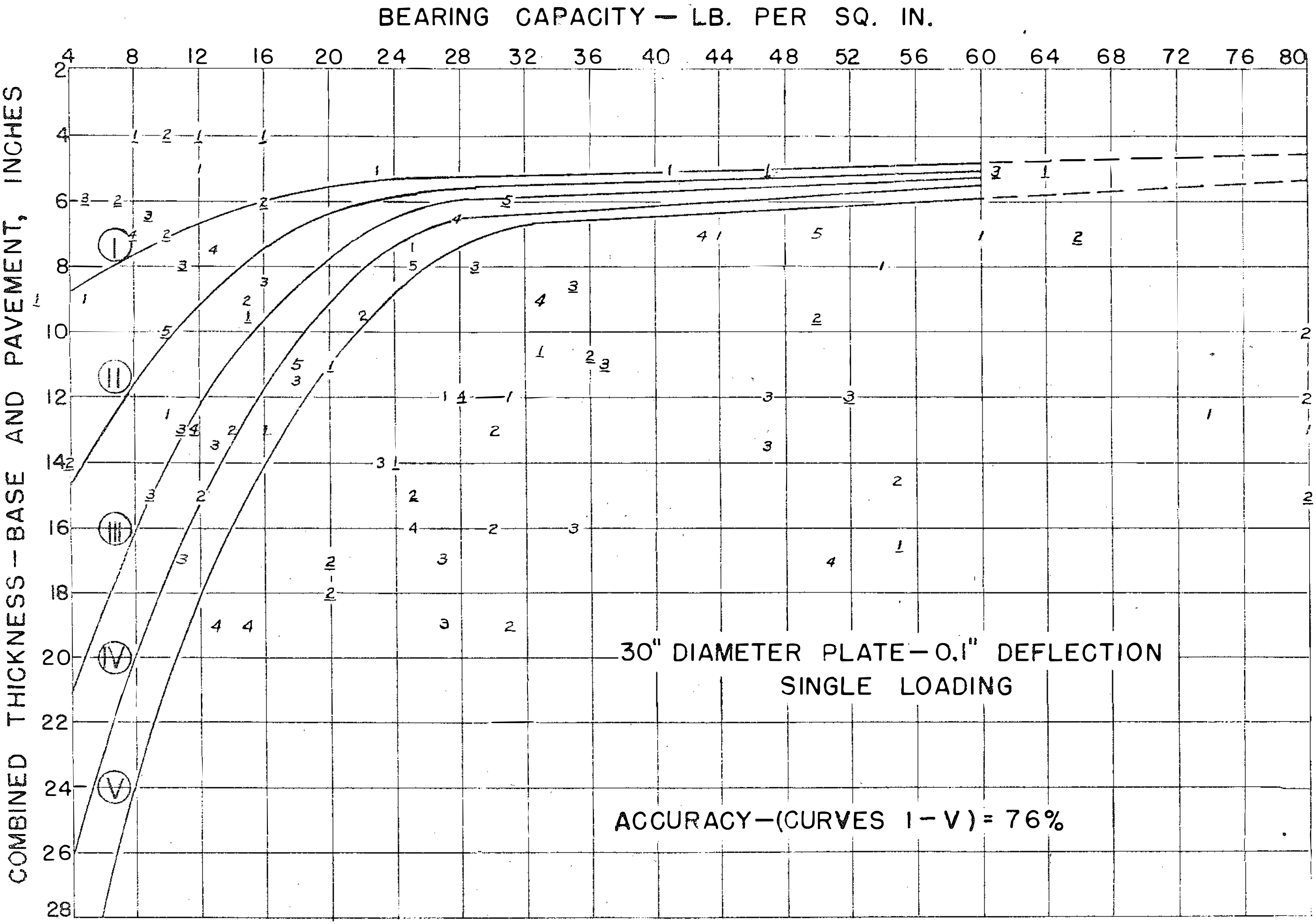

BEARING CAPACITY — LB. PER SQ. IN.
COMBINED THICKNESS - BASE AND PAVEMENT, INCHES
30" DIAMETER PLATE — 0.1" DEFLECTION
SINGLE LOADING
ACCURACY—(CURVES I – V) = 76%

Fig. 13

(2) For maximum protection against frost action, there should be not more than 8 percent passing the No. 200 sieve.

In Table 7, there is a list of the base samples that had a P.I. greater than six. Of the 27 samples, 13 were from good sections, while 14 were from base failures. For each location, the thickness of the existing mat was checked against that required by the curves in Figs. 7, 8, 9, 12 and 13. Nothing of significance was noted. However, at only nine locations was the mat thickness within three inches (plus or minus) of that required by the curves. In addition, of the 27 samples only seven had as much as 10 percent of the sample passing the No. 40 sieve.

In Table 8, all base samples that contained as much as 9 percent passing the No. 200 sieve have been listed. There are 13 from sections with base failures, and 22 from good sections. These sample locations were also checked against the five sets of curves, but again there was no significant trend noted. Only 15 had a mat thickness plus or minus 3 inches of that indicated by the curves.

The densities and moisture contents of the bases were checked. Only 20 of 179 had a moisture content in excess of ten, and of these 11 were at good locations. There were 47 with a density less than 120 lb. per cu. ft., of which 25 were determinations from good locations. Future analyses in greater detail may reveal significant effects of base moistures and densities despite lack of control of the base density determinations.

No quality, strength, or laboratory relative density determinations have been conducted on the base samples. However, approximately 15 pounds of the original base material is available for future analyses.

Relationship between Test Results

Numerous investigators have attempted to estimate the relationship existing between the several proposed methods for evaluating subgrade

TABLE 7

LIST OF BASE SAMPLES WITH P.I. GREATER THAN SIX
(Material Passing No. 40 Sieve)

Location Number	Pavement Performance	Mat Thickness (inches)	P.I.	Percent Passing No. 200 Sieve	Percent Passing No. 40 Sieve
570	No Base Failure	18.0	7	2	5
577	Base Failure	8.0	7	8	17
616	Base Failure	7.0	7	4	5
625	No Base Failure	34.0	7	6	11
639	Base Failure	10.0	7	5	9
514	Base Failure	9.0	8	5	7
572	Base Failure	10.0	8	3	5
581	No Base Failure	10.0	8	23	26
596	No Base Failure	10.0	8	8	15
597	No Base Failure	15.0	8	5	7
598	No Base Failure	18.0	8	6	10
604	No Base Failure	17.0	8	0	3
610	Base Failure	6.0	8	2	7
532	Base Failure	13.0	9	3	5
543	No Base Failure	10.0	9	3	5
582	No Base Failure	12.0	9	12	13
594	No Base Failure	19.0	9	2	3
611	No Base Failure	12.0	9	4	7
529	Base Failure	13.0	10	2	4
603	Base Failure	8.5	10	6	7
670	No Base Failure	8.5	10	0	2
650	Base Failure	9.5	11	4	6
618	Base Failure	9.5	13	7	8
515	Base Failure	8.0	15	5	9
531	No Base Failure	34.0	15	20	23
579	Base Failure	8.0	16	9	12
627	Base Failure	9.0	16	3	5

TABLE 8

LIST OF BASE SAMPLES WITH MORE THAN 8 PERCENT PASSING THE NO. 200 SIEVE

Location Number	Pavement Performance	Mat Thickness (inches)	Percent Passing No. 200	P.I. (Material Passing No. 40 Sieve)
513	Base Failure	13.0	9	6
517	No Base Failure	22.0	9	0
526	No Base Failure	27.0	9	6
551	No Base Failure	5.0	9	0
579	Base Failure	8.0	9	16
583	Base Failure	9.0	9	0
588	No Base Failure	10.0	9	0
613	No Base Failure	9.0	9	2
626	No Base Failure	7.5	9	5
638	No Base Failure	13.0	9	3
661	No Base Failure	14.0	9	0
671	Base Failure	16.5	9	0
528	No Base Failure	6.0	10	0
559	No Base Failure	9.0	10	0
560	No Base Failure	17.0	10	0
571	Base Failure	11.0	10	6
672	No Base Failure	13.0	10	0
512	No Base Failure	33.0	11	0
519	No Base Failure	19.0	11	0
527	Base Failure	10.5	11	0
564	Base Failure	11.0	11	3
578	No Base Failure	7.0	11	5
599	Base Failure	20.5	11	6
635	Base Failure	12.0	11	0
523	No Base Failure	17.0	12	0
561	Base Failure	9.0	12	0
582	No Base Failure	12.0	12	9
676	No Base Failure	7.5	12	0
544	No Base Failure	12.5	13	0
506	No Base Failure	14.0	14	6
678	Base Failure	12.0	15	0
680	Base Failure	---	16	2
679	No Base Failure	7.5	19	0
595	Base Failure	9.5	21	0
581	No Base Failure	10.0	23	8

support (20), (21).

The relationship between the minimum laboratory and minimum field CBR was analyzed by dividing the results into four groups on the basis of relative conditions of moisture and density. These are contained in Fig. 14 which shows no dependable relationship for any of the groups. This would be anticipated for all but Group 1. This latter group contains all situations at which the field moisture contents and densities were comparable to those in the laboratory. Unfortunately, there were only eleven in this group.

The inconsistencies between laboratory CBR and field CBR conditions of moisture and density in the CBR tests indicate conclusively that the laboratory technique employed in this study is in need of modifications. Research on the effect of a laboratory compaction and the soaking period taking cognizance of the work by the U. S. Engineers at Vicksburg (19) has been initiated as a second phase of this investigation of soils and their relation to flexible pavements in Kentucky.

The minimum field CBR was plotted versus the average North Dakota Cone bearing value in Fig. 15. It will be noted that for cone bearing values up to 400 and CBR's up to ten, there is a good relationship. Above these values, there is considerable variation in bearing values.

Nothing more than a "trend" appears to be indicated by the plot of the field CBR versus the bearing capacity for the 12-inch plate, 0.1-inch deformation, and single loading (Fig. 16). Extreme variations occur in all ranges of bearing values.

A trend is indicated by the plot of the North Dakota Cone bearing value versus the bearing capacity for the 12-inch plate, 0.1-inch deformation, single loading, shown in Fig. 17. Even so, there is considerable variation throughout the range of bearing values.

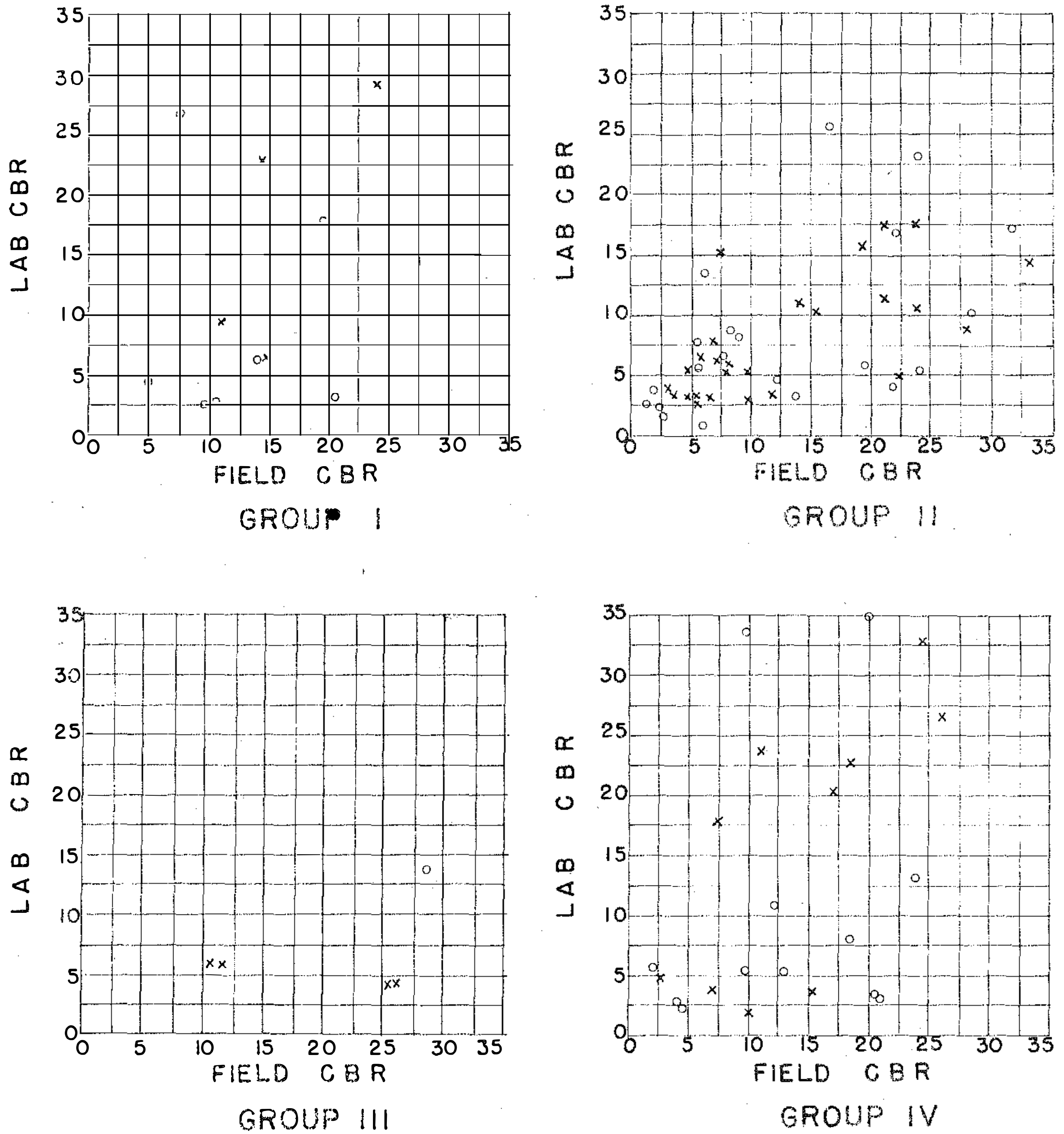


Fig. 15

Group I. Samples whose density is within plus or minus three percent and moisture content within plus or minus two percent.
Group II. Samples whose moisture content is within plus or minus two percent.
Group III. Samples whose density is within plus or minus three pounds.
Group IV. Samples not included in previous groups.

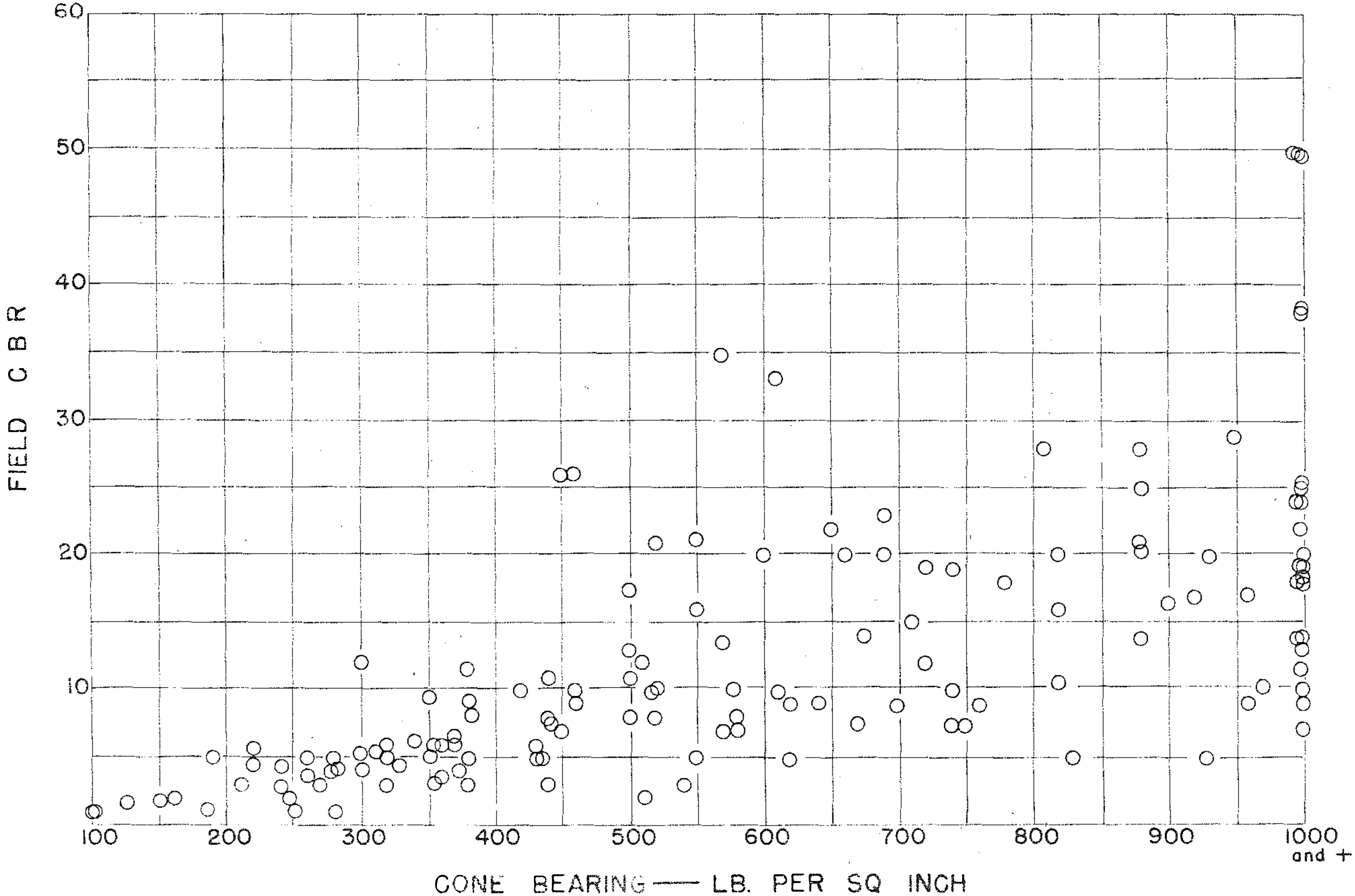
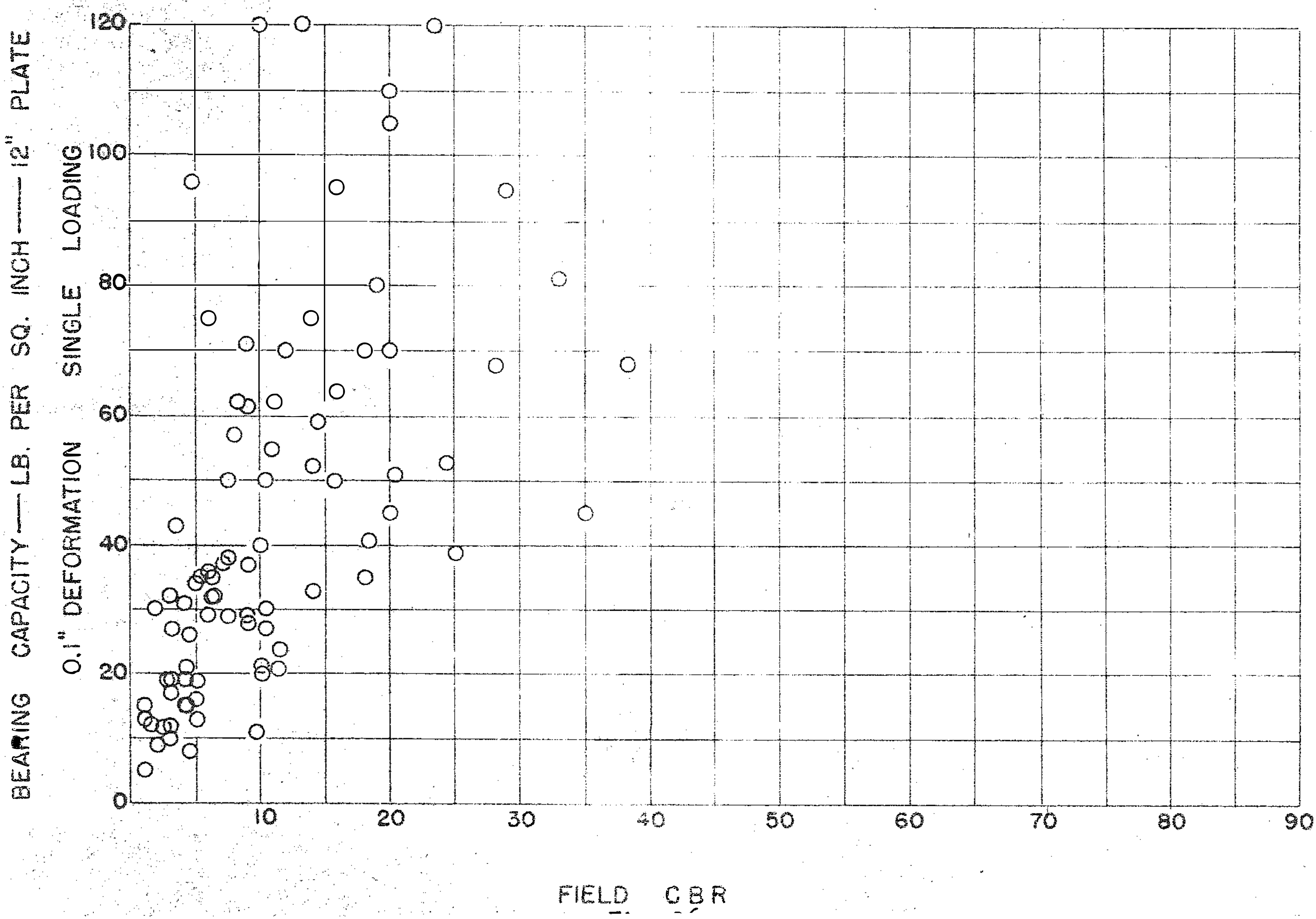
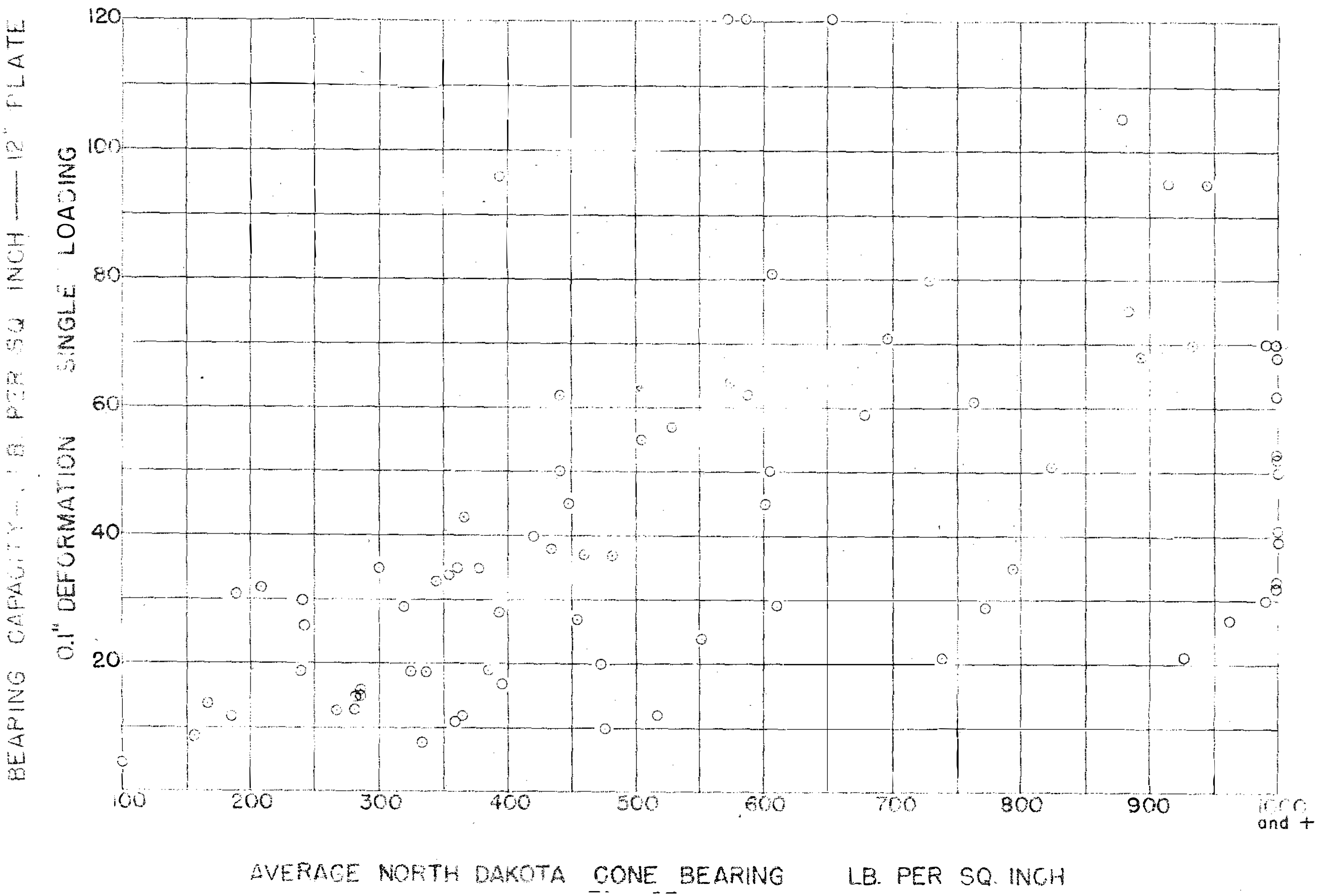

60
50
40
30
20
10
FIELD C B R
100
200
300
400
500
600
700
800
900
1000 and +
CONE BEARING —— LB. PER SQ INCH

Fig. 15


BEARING CAPACITY — LB. PER SQ. INCH — 12" PLATE
0.1" DEFORMATION SINGLE LOADING
0
20
40
60
80
100
120
10
20
30
40
50
60
70
80
90
FIELD C B R


BEARING CAPACITY — LB. PER SQ. INCH — 12" PLATE
0.1" DEFORMATION SINGLE LOADING
120
100
80
60
40
20
100
200
300
400
500
600
700
800
900
1000 and +
AVERAGE NORTH DAKOTA CONE BEARING LB. PER SQ. INCH

RESULTS

The necessary approximations in the determination of traffic make any traffic value a variable. The Equivalent Wheel Load method of computation proved sufficiently reliable in differentiating thickness versus bearing to permit an overall accuracy of from 77 to 83 percent, whereas comparable determinations without regard to traffic exceeded 70 percent in only one instance. Moreover the Equivalent Wheel Load value, as opposed to specific wheel loads measured by loadometer, is very useful in projecting traffic, particularly with a limited number of loadometer stations.

The variation in seasonal moisture content was less than 2 percent for 33 of 81 locations. In 33 percent of 149 locations, the field moisture content was greater than the plastic limit, and at 27 percent of these locations it was within 2 percent of the plastic limit. At 48 percent of 124 locations the field density was greater than 100 percent maximum density (A.S.T.M. D698).

With regard to moisture contents of the field and laboratory CBR tests, at 51 percent of 147 locations the field moisture content was greater than the corresponding laboratory moisture content. At 41 percent of the locations the two moisture contents were within plus or minus 2 percent of each other. At 81 percent of 128 locations the laboratory CBR density was greater than field density. In 59 percent of the cases field density was within plus or minus 3 percent of maximum density.

For field and laboratory CBR, the analysis of bearing ratios for each increment of penetration showed little variation in percentage accuracy of pavement thickness curves. The minimum CBR value was selected as the best <u>comparative</u> value between the two tests.

An analysis of the bearing plate tests showed that for 34 percent of 68 locations there was a good agreement with the perimeter-area ratio theory, and in an additional 16 percent the agreement was reasonably good. Several reasons for the disagreement in the remaining 50 percent of the cases were established, but in no specific instance could the disparities be explained.

Comparisons among the unmodified bearing values obtained from the different methods of test resulted in:

(1) No definite relationship between laboratory CBR and field CBR.

(2) Excellent agreement between field CBR and cone up to values of 400 for the cone and 10 for the CBR. Thereafter, the divergence became progressively greater with no dependable relationship existing.

(3) Trends in relationships between plate loading and both the field CBR and the cone.

No method of test produced better than 83 percent accuracy in predicting performance, unless some regard is given to extenuating circumstances that are reasonable and probably valid. Undoubtedly there were a number of circumstances represented at locations where tests were made that have no counterpart in newly designed and constructed pavements. For example, adverse drainage to the extent of ponded ditches and frequently flooded roadways are factors beyond usual design requirements. Similarly, widened pavements offer possibilities for failure which would not be represented in new construction projects. The new base may not be of the same quality or even the same type as the old.

With this in mind, all locations were reviewed individually by reference to field notes and photographs, and in many cases these were supplemented by new notes taken at the time test locations were inspected again. During this review, no attention whatsoever was given to test results in order to avoid elimination or modification of results on the basis of anything but field conditions.

In 12 instances the probable cause of failure of the base to support the pavement was traced to adverse drainage, and there were five locations where widening of the pavement was considered so influential that performance of the road as a whole was not represented by performance at the spots tested. Seven other samples were judged non-representative for various reasons. Accordingly, analytical data were modified either by elimination of the test results entirely or by a change in the pavement performance rating. Table 9 shows the locations affected the comparative degree of failure, the reasons for change, and the actions taken.

With these modifications, the data were used in plotting new sets of curves for pavement thickness versus bearing value as determined by the

TABLE 9

SAMPLES INVOLVED IN MODIFICATION OF DATA

Sample No.	Extent of Failure	Drainage	Action Taken	Cause for Change
535	Slight	Very Poor	Eliminated	Adverse Drainage
539	Edge	Very Poor	Eliminated	Adverse Drainage
542	Bad	Very Poor	Eliminated	Adverse Drainage
557	Bad	Very Poor	Eliminated	Adverse Drainage
589	Slight	Fair	Eliminated	Section Sampled did not represent Failed Portion of Road
599	Bad	Very Poor	Eliminated	Adverse Drainage
603	Edge	Good	Changed to Good Section	Widening Failure, Center Section Good
610	Edge	Poor	Eliminated	Adverse Drainage
616	Slight	Good	Changed to Good Section	Surface Failure
618	Bad	Poor	Eliminated	Adverse Drainage
622	Slight	Poor	Eliminated	Adverse Drainage
630	Edge	Fair	Eliminated	Settlement of Widened R.R. Bed
635	Moderate	Very Poor	Eliminated	Adverse Drainage
637	Moderate	Poor	Eliminated	Adverse Drainage
639	Edge	Poor	Eliminated	Adverse Drainage
640	Slight	Fair	Changed to Good Section	Surface Failure
650	Edge	Fair	Changed to Good Section	Widening Failure, Center Section Good
652	Bad	Very Poor	Eliminated	Adverse Drainage
654	Edge	Fair	Changed to Good Section	Widening Failure, Center Section Good
657	Edge	Good	Changed to Good Section	Widening Failure, Center Section Good
658	Edge	Good	Changed to Good Section	Widening Failure, Center Section Good
665	Moderate	Fair	Eliminated	Fill Settlement
671	Moderate	Poor	Eliminated	Fill Settlement
678	Moderate	Poor	Eliminated	Rock Asphalt Placed too thin

several types of loading tests. In every case, the object was to draw relationships which would produce the highest degree of accuracy for the test results in predicting pavement performances. However, in the high bearing value range where the control was very indefinite due to a scarcity of samples, required thicknesses were arbitrarily reduced. Curves for the minimum field CBR value versus mat thickness are shown in Fig. 18. Traffic groups are the same as those used in Fig. 8 and outlined in Table 2. Thicknesses corresponding to given CBR values in this set of curves for the modified results are only slightly less than those for the unmodified results plotted in Fig. 8. However, the overall percentage of accuracy was raised from 84 to 90 percent.

In Fig. 19, the North Dakota Cone Bearing values are plotted against total mat thicknesses in accordance with the modified set of results. Here again, the accuracy (92 percent) of the revised data increased greatly over the accuracy (79 percent) produced by the unmodified results plotted in Fig. 9, yet the total mat thickness corresponding to any certain bearing value was reduced only a slight amount. For the lower ranges of North Dakota Cone bearing values the thicknesses were practically unaffected, while in the higher ranges the thicknesses were lightened about one inch.

Thicknesses designated by plate bearing values are shown in the modified form in Fig. 20 and Fig. 21. These are in contrast to the unmodified curves in Fig. 12 and Fig. 13, respectively. Comparisons among these show that thicknesses required for certain bearing values have been reduced in amounts exceeding 12 inches for the very low bearing values; however, in the range of plate bearing values above 30 lb. per sq. in. the required thicknesses were sometimes increased a slight amount by the modifications. Most of these did not exceed a one inch increase, although there were instances of very high bearing values where the

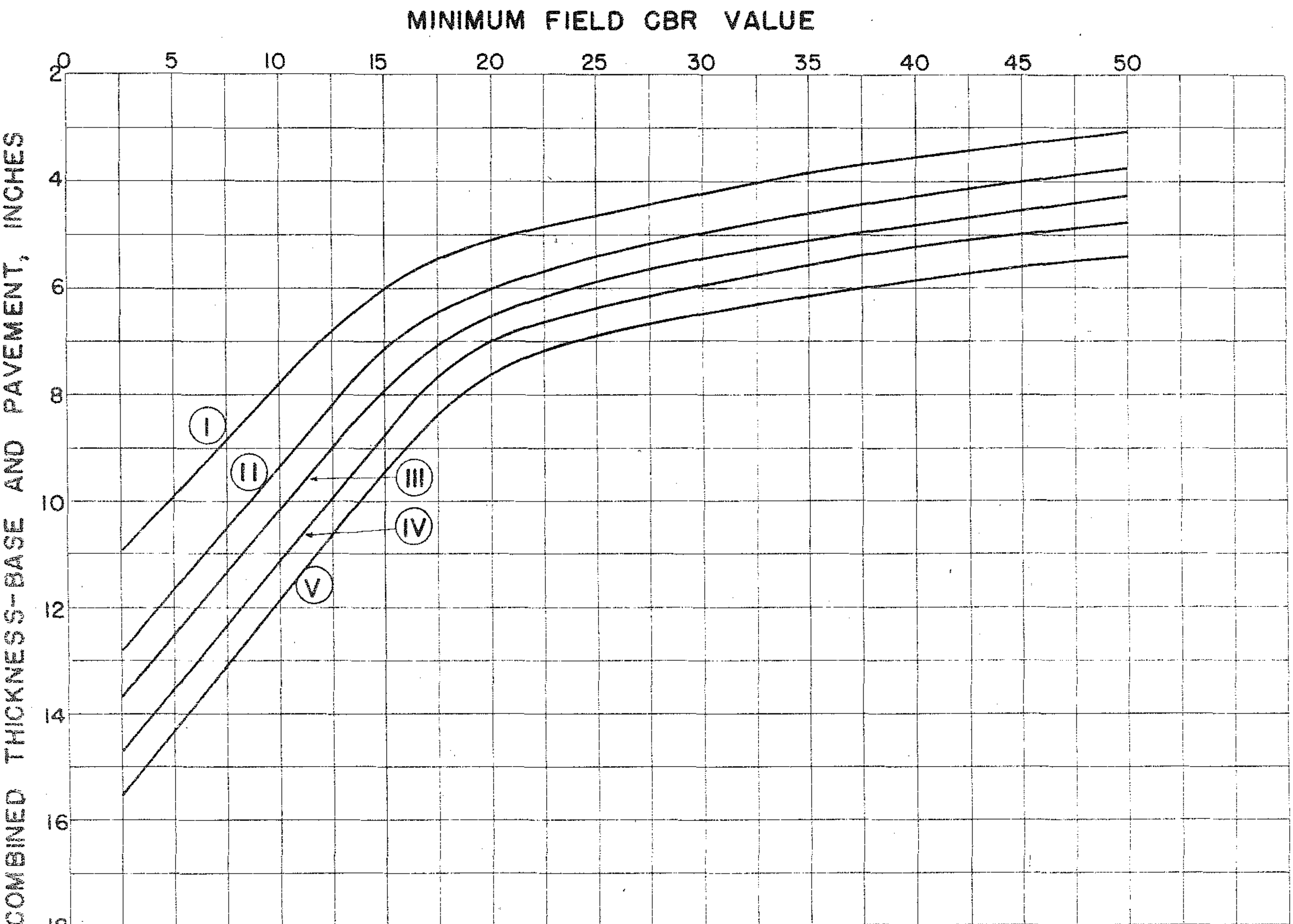

MINIMUM FIELD CBR VALUE
0
5
10
15
20
25
30
35
40
45
50
COMBINED THICKNESS-BASE AND PAVEMENT, INCHES
2
4
6
8
10
12
14
16
18
I
II
III
IV
V

Fig. 18

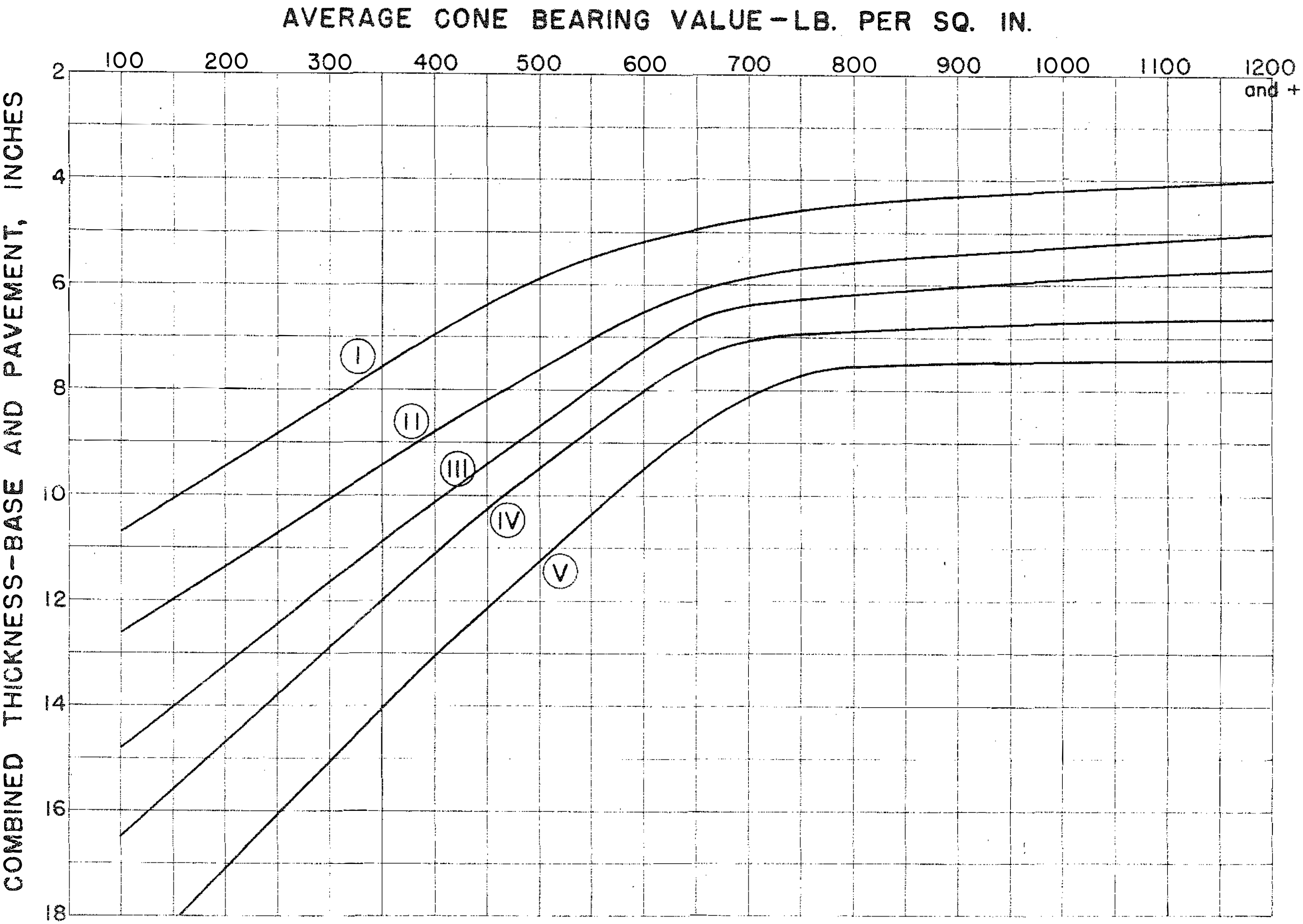

AVERAGE CONE BEARING VALUE - LB. PER SQ. IN.
100
200
300
400
500
600
700
800
900
1000
1100
1200 and +
COMBINED THICKNESS-BASE AND PAVEMENT, INCHES
2
4
6
8
10
12
14
16
18
I
II
III
IV
V

Fig. 19

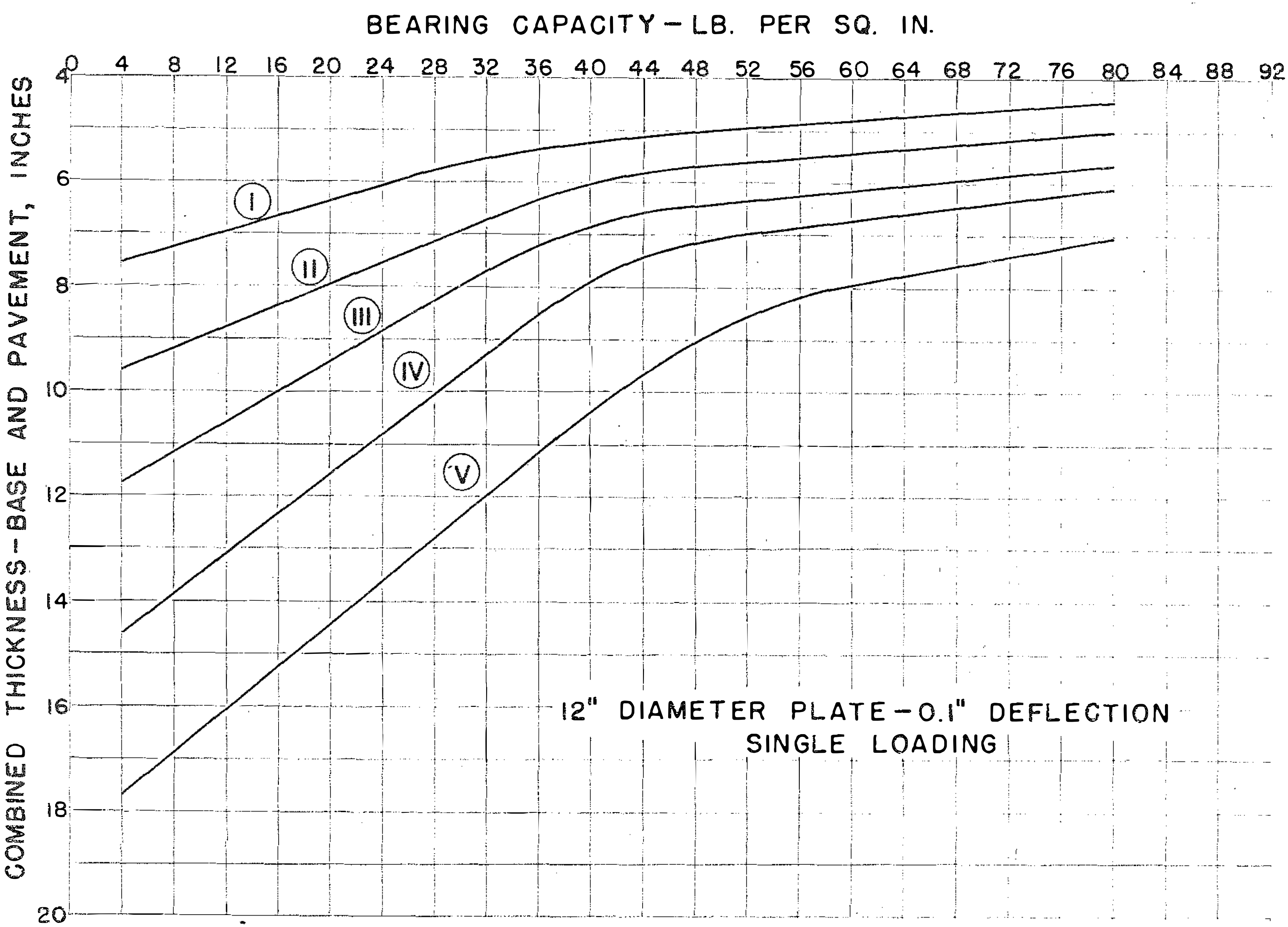

BEARING CAPACITY - LB. PER SQ. IN.
0 4 8 12 16 20 24 28 32 36 40 44 48 52 56 60 64 68 72 76 80 84 88 92
COMBINED THICKNESS - BASE AND PAVEMENT, INCHES
4 6 8 10 12 14 16 18 20
I
II
III
IV
V
12" DIAMETER PLATE - 0.1" DEFLECTION
SINGLE LOADING

Fig. 20

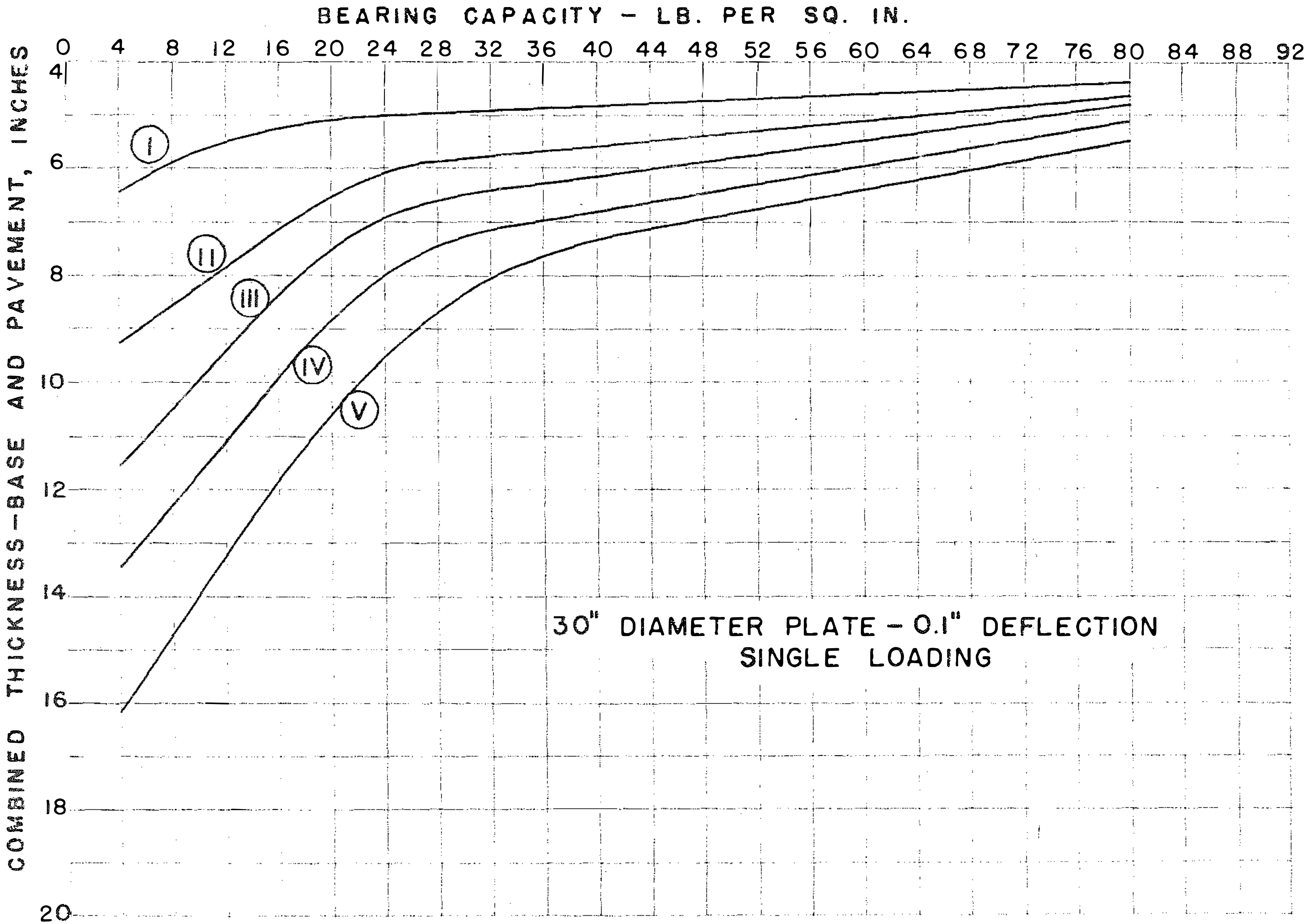

BEARING CAPACITY - LB. PER SQ. IN.
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
76
80
84
88
92
COMBINED THICKNESS-BASE AND PAVEMENT, INCHES
4
6
8
10
12
14
16
18
20
I
II
III
IV
V
30" DIAMETER PLATE - 0.1" DEFLECTION
SINGLE LOADING

Fig. 21

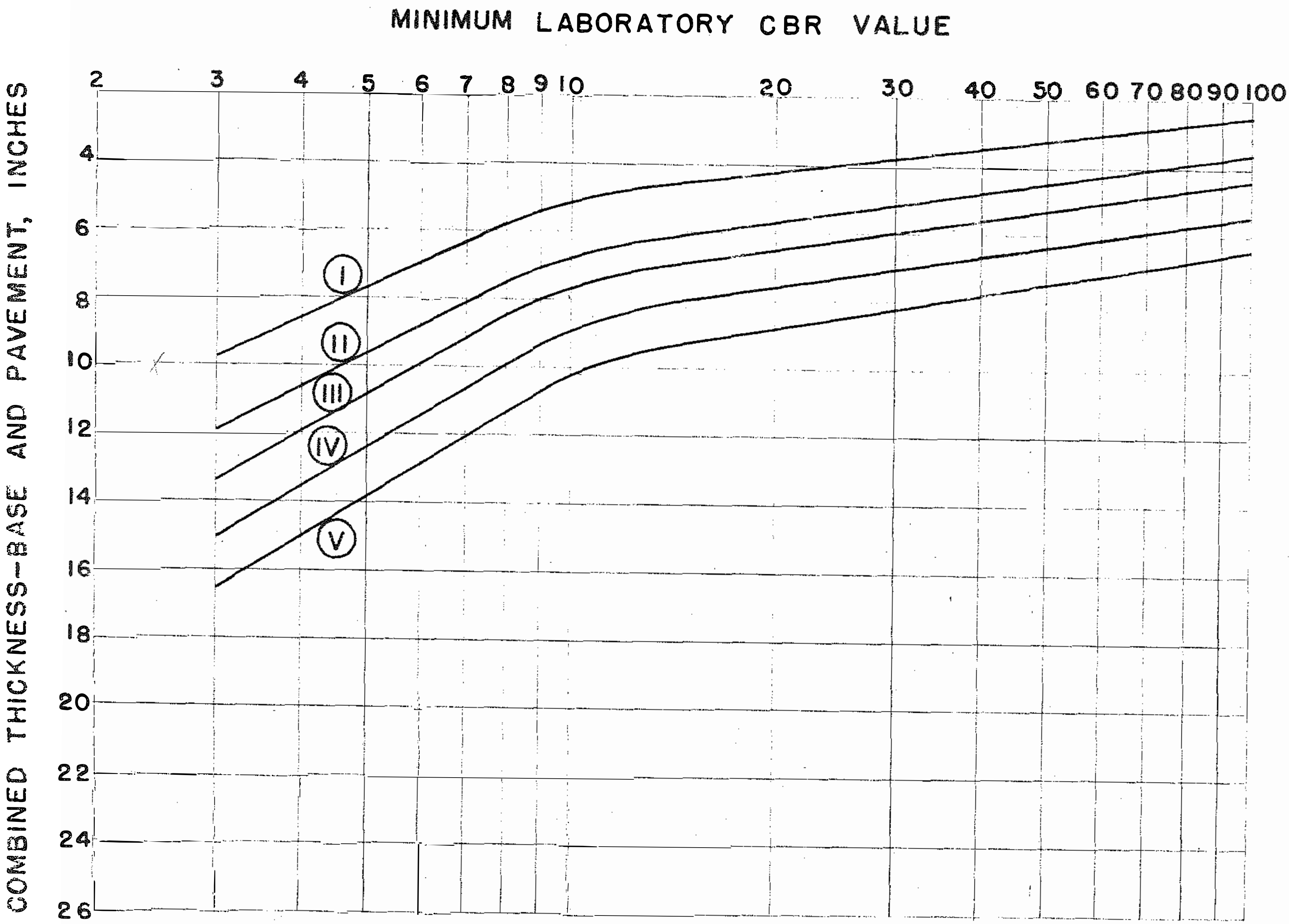

MINIMUM LABORATORY CBR VALUE
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
COMBINED THICKNESS-BASE AND PAVEMENT, INCHES
4
6
8
10
12
14
16
18
20
22
24
26
I
II
III
IV
V

Fig. 22

increase ran up to about two inches. The percentage of accuracy for the 12-inch plate was raised from 78 percent to 90 percent, and for the 30-inch plate it was increased from 76 percent to 89 percent because of these modifications in data.

Curves showing the relationship between minimum laboratory CBR and combined pavement thickness are plotted in Fig. 22 in accordance with the modified data. Original curves without modification were presented in Fig. 7. The effect of modification here was reduction or at least no increase in the indicated thickness required for subgrades having certain laboratory CBR values. Most pronounced changes were in the low ranges of CBR (below 4) where one reduction in indicated thickness was 5 inches and would have been greater had the curves in Fig. 22 been carried below CBR values of 3. There were also considerable reductions in thickness corresponding to CBR's above 40, although these did not exceed a change of 2 inches in thickness. Throughout the middle range between 4 and 20 CBR values, the reductions in thickness casued by the modifications were usually less than one inch. That is the range where data are most abundant and, of course, the range within which most of the soils fall. The percentage of accuracy for this method was increased from 74 percent to 82 percent as a result of the modifications.

CONCLUSIONS

Each of these five sets of curves in Figs. 18 to 22 may be considered a design in itself. Each test has a method for determining a bearing value of the subgrade. The feasibility of using any of the three field test methods for a design is questionable. It is recognized that the field test methods allow a greater accuracy for predicting performance, however, the difficulty of conducting field tests under conditions prior to design makes them impractical. Thus, it is desirable to perfect a laboratory method of test that will reflect conditions in the field and predict performance to the degree of accuracy that can be obtained by the field tests.

The curves for the minimum laboratory CBR values in Fig. 22 represent the best basis for a design at the present time. Because of the limited number of samples representing CBR's lower than 3, and the critical nature of soils having such extremely low supporting power, designs should not be made on extensions of the curves in Fig. 22 to CBR's lower than 3. These are materials that should be avoided (removed if possible), but if design must be made against their use, then the curves in Fig. 7 provide a much greater factor of safety.

Mention should also be made of the fact that the laboratory CBR, being the result of a test not performed on the soil exactly as it existed beneath the pavement, could reasonably be influenced more by modifications than any of the other test results. However, the changes brought about by the modified data did not bring this out.

Modification of compaction and soaking treatments in the present laboratory CBR test procedure could possibly produce densities and moisture contents that more nearly represent those of the soil in the subgrade. If that could be accomplished, better correlation between the laboratory CBR test and field performance might be obtained. Investigation of the test procedure from that standpoint is proposed as a sequel to this study.

ACKNOWLEDGMENTS

The general supervision and guidance of Mr. L. E. Gregg, Associate Director of the Research Laboratory were of inestimable value in this investigation. For his full understanding of all requirements and for his cooperation in preparing this paper the writers are materially indebted.

The authors wish to acknowledge the cooperation and assistance of many individuals in the Kentucky Department of Highways including: Mr. D. H. Bray, Chief Engineer, who as Director of Design in 1947 fostered this investigation through requests, suggestions, and interest in data pertaining to flexible pavement design; Mr. A. O. Neiser, Assistant Director of Design; Messrs. R. E. Bagby, Director and J. R. Pulliam, Engineer of traffic surveys in the Division of Planning; Messrs. J. A. Bitterman, Engineer of Tests and O. R. Threlkold, Soils Engineer in the Testing Laboratory; and Mr. Hack Reed of the Reproduction Section. The cooperation of District Engineers and their personnel aided materially in the accomplishment of field operations.

Mr. Ellis G. Williams, Assistant Research Engineer, was responsible for the compilation and much of the analysis of traffic data as well as the planning and performance of a large part of the laboratory tests.

The writers wish to acknowledge the assistance and wholehearted cooperation of the following individuals: Mr. Zack Pulliam, Jr., for his part in the field and laboratory work; Messrs. Arnold D. Baker, Paul E. Brawner, William S. Davis, Robert C. Deen, William A. Egalite, John L. Elmore, and Ellis Kayse for their cooperation in the field sampling and laboratory testing; Messrs. Othniel Alsop, Dwight C. Bray, Allen W. Daugherty, Harvey J. Field, Jr., J. M. Heidenreich, Roy Hixson, Donald C. Newberry, Jr., Willard N. Stooke, and George C. Wilson for their work in the laboratory testing; Mr. Allie C. Peed, Jr. for his photographic work; and all other personnel in the Research Laboratory for their full cooperation throughout.

The writers wish to extend their sincerest and deepest appreciation to Mrs. Ernest King, Mrs. Frances Keesee and Miss Margery McCabe for their untiring work in the preparation of this manuscript.

REFERENCES

(1) Highway Research Board Subcommittee on Flexible Pavement Design, "Progress Report", Proceedings, Highway Research Board, Vol. 25 (1945).

(2) Porter, O. J., "Foundations for Flexible Pavements", Proceedings, Highway Research Board, Vol. 22 (1942).

(3) Russell, I. E. and Olinger, D. T., "Wyoming Methods of Flexible Pavement Design", Proceedings, Highway Research Board, Vol. 27 (1947).

(4) Bail, E. B., "A Method for Estimating Required Thickness of Flexible Base", Proceedings, Highway Research Board, Vol. 27 (1947).

(5) Livingston, R. E., "Design of Flexible Bases", Proceedings, Highway Research Board, Vol. 27 (1947).

(6) Swanberg, J. H. and Hansen, C. C., "Development of a Procedure for the Design of Flexible Bases", Proceedings, Highway Research Board, Vol. 26 (1946).

(7) Boyd, Keith, "An Analysis of Wheel Load Limits as Related to Design", Proceedings, Highway Research Board, Vol. 22 (1942).

(8) Housel, W. S., "The Design of Flexible Surfaces", Proceedings, Twenty-third Annual Highway Conference, University of Michigan, February 16-18, 1937.

(9) "Design of Flexible Pavements Using the Tri-Axial Compression Test", Highway Research Board, Bulletin No. 8, 1947.

(10) Barber, E. S., "Tri-Axial Compression Test Results Applied to Flexible Pavement Design", Public Roads, Vol. 25, No. 1, September 1947.

(11) Campen, W. H. and Smith, J. R., "Use of Load Tests in the Design of Flexible Pavements", Symposium on Load Tests of Bearing Capacity of

Soils, Fiftieth Annual Meeting, American Society for Testing Materials, June 16-20, 1947.

(12) "Procedure for Determination of Thicknesses of Flexible Type Pavements", Manual No. 1, Bureau of Yards and Docks, Navy Department, March 1943.

(13) "Report of Committee on Flexible Pavement Design", Proceedings, Highway Research Board, Vol. 23 (1943).

(14) Campen, W. H. and Smith, J. R. "An Analysis of Field Load Bearing Tests Using Plates", Proceedings, Highway Research Board, Vol. 24 (1944).

(15) Middlebrooks, T. A. and Bertram, G. E., "Soil Tests for Design of Runway Pavements", Proceedings, Highway Research Board, Vol. 22 (1942).

(16) Palmer, L. A., "The Evaluation of Wheel Load Bearing Capacities of Flexible Pavements", Proceedings, Highway Research Board, Vol. 26 (1946).

(17) Grumm, F. T., "Designing Foundation Courses for Highway Pavements and Surfaces", California Highways and Public Works, March 1942.

(18) Bradbury, R. P., "Reinforced Concrete Pavements", Wire Reinforcement Institute, 1938.

(19) Wartime Road Problems Bulletin Number 8.

(20) McLeod, N. W., "Airport Runway Evaluation in Canada", August 1947.

(21) Goldbeck, A. T., "The Problem of Flexible Pavement Design", The Crushed Stone Journal, June 1948.