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

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SYNOPSIS

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 on which laboratory CBR tests were conducted. Undisturbed samples for future tri-axial 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.

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*)[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 formula 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

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

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 the density determination of base and sub-base materials, and field and laboratory sampling and testing of subgrade materials from 185 locations representing 434.6 mi. of road selected on the basis of traffic, performance, type of subgrade soil, and types of base and surface construction. 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, ten special loadometer stations were set up and operated in the fall of 1947. The data thus obtained, combined with those from 15 routine stations measured in early summer of 1947, furnished the basis for the analysis of traffic conditions.

Before 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 was 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 some of the former locations were visited for moisture content sampling at the

Figure 1. Base Failure

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. Figure 1 is a photo of a typical failure. Figure 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 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.

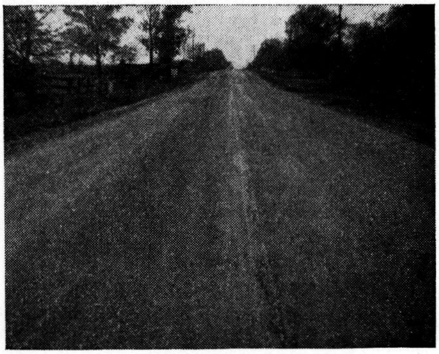

Figure 2. Surface Failure

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 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 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.

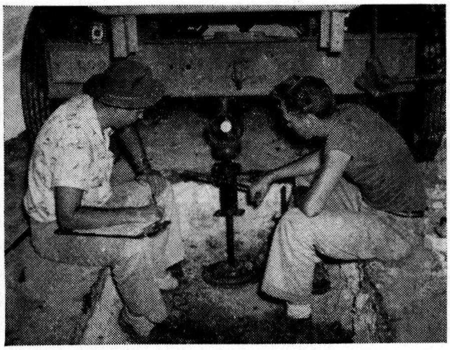

Figure 3. Twelve-inch Diameter Plate Test in Progress

After the excavation was complete, the truck was backed over the hole and the ends of the I-beam were jacked as shown in Figure 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 arrangement. The 10,000-lb. 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 Figure 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 Figures 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

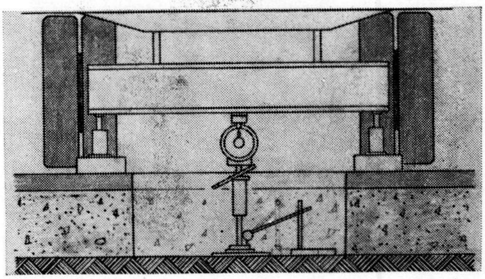

Figure 4. Diagrammatic Sketch of 12-in. Diameter Plate Set Up

dial support. Campen and Smith (*14*) have made some measurements for heavy loads with 12-in. 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-in.

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 4-, 6-, 9-, and 12-in. diameter circular rigid plates, and the maximum deflections were those that could be obtained for the 12-in. 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 min.

As to the repetitional loading, time permitted only three repetitions rather than the five that have been recommended by the

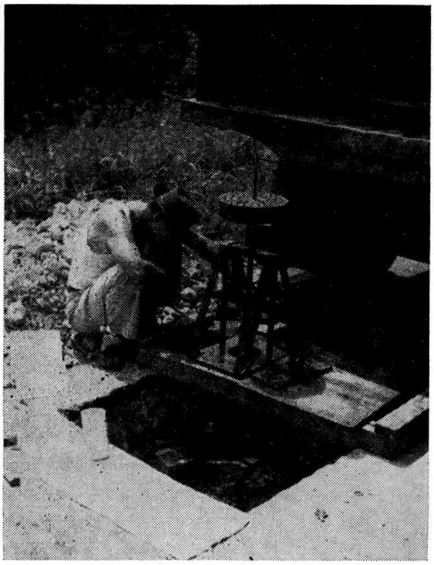

Figure 5. North Dakota Cone Test in Progress

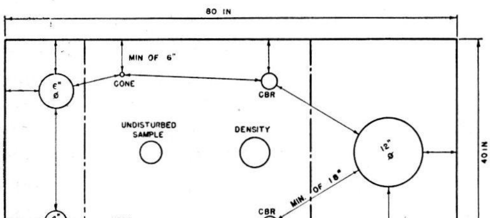

Figure 6. Layout of Test Locations in Hole

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. Figure 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 Figure 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.

Upon conclusion of the sampling and testing, the holes were backfilled 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 hr. 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—A total of 434.6 miles of pavement were studied. Test locations (185 in all) averaged 2.34 mi. apart. The following tabulation describes the nature of the locations and tests made:

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 Samples	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 values of the subgrade were plotted against the total thickness above the subgrade. A traffic value for each sample was noted on the plot.

TABLE 1
LIST OF FACTORS APPLIED TO WHEEL LOADS IN CALCULATING EQUIVALENT 5000-POUND WHEEL LOADS

Wheel Load	Factor[a]	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

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

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 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.

DISCUSSION OF THE FACTORS STUDIED AND TESTS MADE

Traffic—The available traffic data included loadometer measurements from nine 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-lb. 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*).

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

The EWL value is essentially a means of including in the traffic factor the weighted effect of various 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 in Table 4 there is a summary of the data from 82 locations at which moisture measurements were made during at least two seasons of 1948.

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

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.

Table 5 summarizes 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.

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

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

TABLE 5

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

PRA Classification	Description	Plastic Limit (A)		Optimum Moisture Content (B)		Laboratory CBR Moisture Content (Entire Sample) (C)	
		No of Locations	Percent of Total Samples	Percent No. of Locations	of Total Samples	No. of Locations	Percent of total Samples
A-1	Total number of Locations	0		2		1	
	Location where FMC was equal to or greater than (A, B, or C)	0	0	0	0	1	100
	Locations where FMC ±2% was equal to (A, B, or C)	0	0	0	0	0	0
A-2	Total number of Locations	5		10		8	
	Location where FMC was equal to or greater than (A, B, or C)	0	0	4	40	4	50
	Locations where FMC ±2% was equal to (A, B, or C)	1	20	4	40	2	25
A-2-4	Total number of Locations	2		2		3	
	Locations where FMC was equal to or greater than (A, B, or C)	0	0	1	50	2	67
	Locations where FMC ±2% was equal to (A, B, or C)	1	50	1	50	1	33
A-4	Total number of Locations	71		73		69	
	Locations where FMC was equal to or greater than (A, B, or C)	25	35	27	37	34	49
	Locations where FMC ±2% was equal to (A, B, or C)	18	25	36	49	31	45
A-4-5	Total number of Locations	0		1		1	
	Locations where FMC was equal to or greater than (A, B, or C)	0	0	0	0	1	100
	Locations where FMC ±2% was equal to (A, B, or C)	0	0	1	100	1	100
A-4-6	Total number of Locations	5		7		7	
	Locations where FMC was equal to or greater than (A, B, or C)	2	40	3	43	4	57
	Locations where FMC ±2% was equal to (A, B, or C)	0	0	2	29	4	57
A-5	Total number of Locations	52		45		39	
	Locations where FMC was equal to or greater than (A, B, or C)	15	29	23	51	18	46
	Locations where FMC ±2% was equal to (A, B, or C)	14	27	21	47	18	46
A-5-6	Total number of Locations	0		0		1	
	Locations where FMC was equal to or greater than (A, B, or C)	0	0	0	0	1	100
	Locations where FMC ±2% was equal to (A, B, or C)	0	0	0	0	0	0
A-5-7	Total number of Locations	12		14		14	
	Locations where FMC was equal to or greater than (A, B, or C)	6	50	9	64	6	43
	Locations where FMC ±2% was equal to (A, B, or C)	5	42	4	29	3	21
A-6	Total number of Locations	1		3		3	
	Locations where FMC was equal to or greater than (A, B, or C)	1	100	2	67	2	67
	Locations where FMC ±2% was equal to (A, B, or C)	0	0	1	33	0	0
A-7	Total number of Locations	1		1		1	
	Locations where FMC was equal to or greater than (A, B, or C)	0	0	1	100	1	100
	Locations where FMC ±2% was equal to (A, B, or C)	1	100	0	0	0	0
Total	Total number of Locations	149		158		147	
	Locations where FMC was equal to or greater than (A, B, or C)	49	33	70	44	74	50
	Locations where FMC ±2% was equal to (A, B, or C)	40	27	70	44	60	41

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

The data indicate that the moisture contents of the field and laboratory CBR test samples were reasonably close. However, Table 6 shows that the densities obtained in the Laboratory CBR test were considerably greater than those from the field. The same was true for the percentage of saturation.

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 10 percent of 128 cases.

TABLE 6
SUMMARY OF RELATIONSHIP BETWEEN THE LABORATORY CBR DENSITY[a] AND PERCENT SATURATION[b] 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

[a] Ratio of unit dry weight of a soil to Standard Proctor Maximum Density.
[b] Ratio of volume of water to volume of voids (where voids is all space not occupied by soil particles)

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 7 indicate very little difference as to which penetration was used. Those developed for the minimum laboratory CBR are shown in Figure 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 4- to 6-in. 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.

TABLE 7
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	78
	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

Field CBR—In Figure 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 7.

The accuracy of these curves was 83 percent,

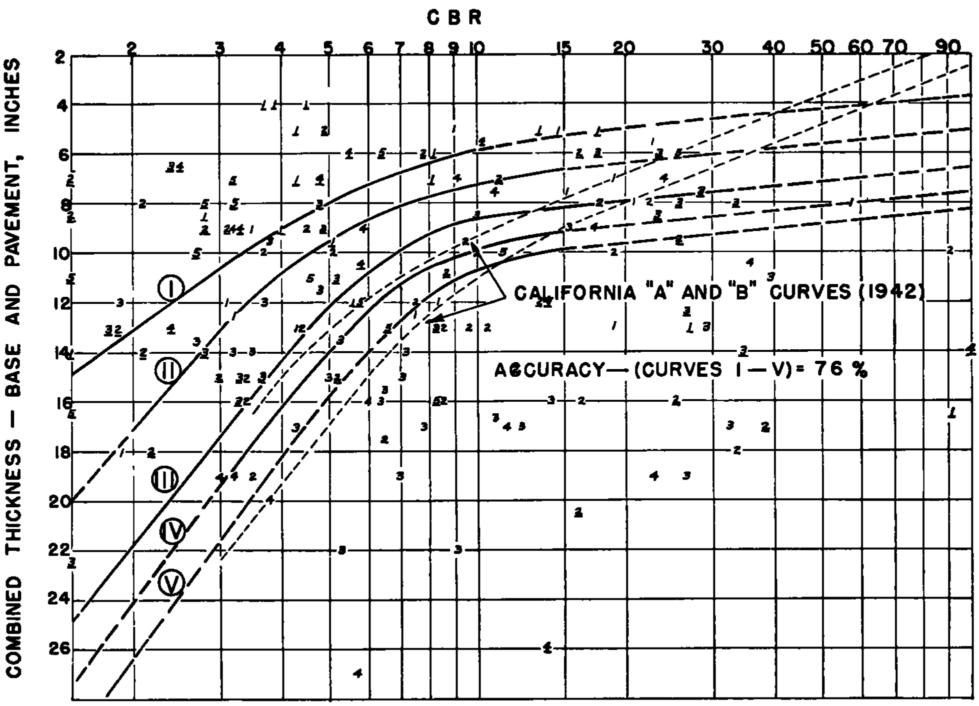

Figure 7. Plot of Minimum Laboratory CBR Values Versus Combined Thickness—Locations are separated into traffic groups as shown in Table 2. Samples in Group 1 are designated by the Arabic numeral 1, etc.

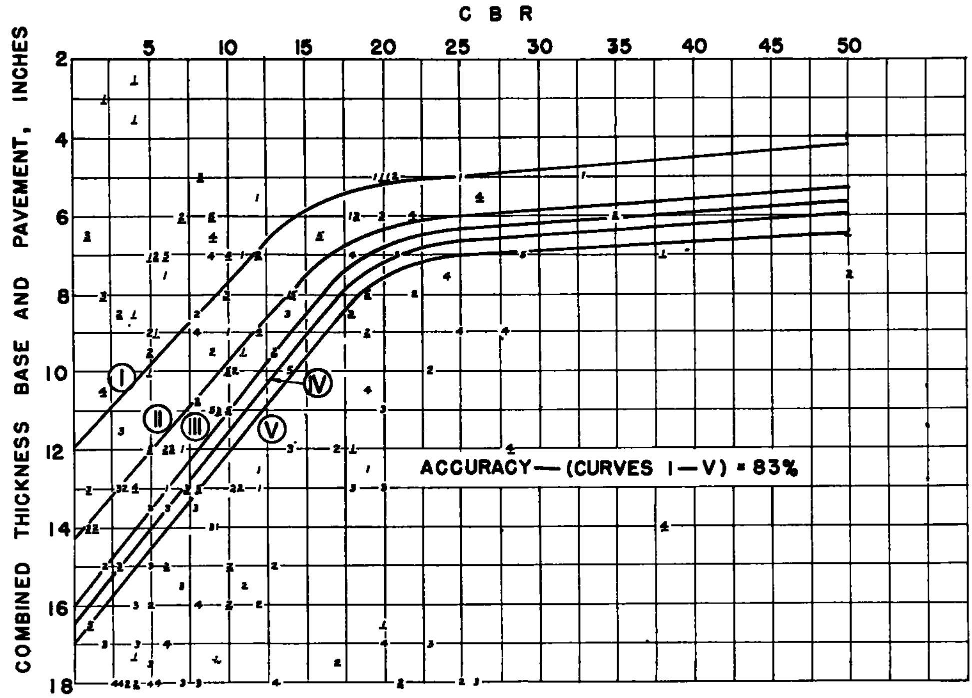

Figure 8. Plot of Minimum Field CBR Values Versus Combined Thickness Above Subgrade—Performance and traffic are plotted in the same manner as in Fig. 7.

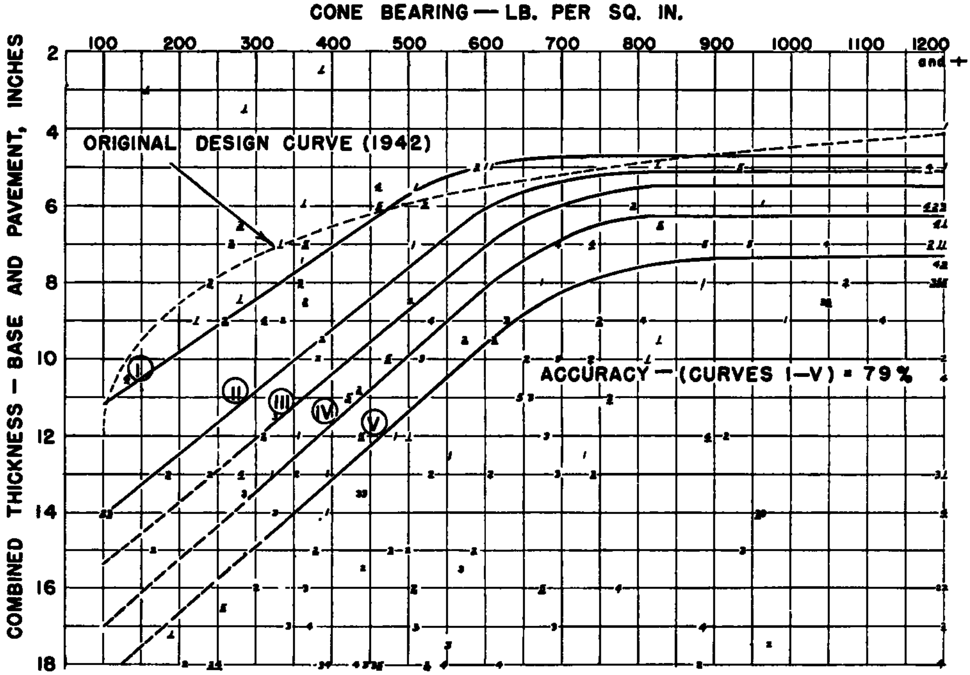

Figure 9. Average North Dakota Cone bearing values are plotted against combined thickness above subgrade. Performance and traffic are shown as in Fig. 7.

six percent higher than for the laboratory CBR data.

North Dakota Cone—The minimum, maximum, 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 Figure 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 in. was the maximum that was available from the data without extensive projection of the curves. The load for each plate at 0.1 in. deformation and a single load-

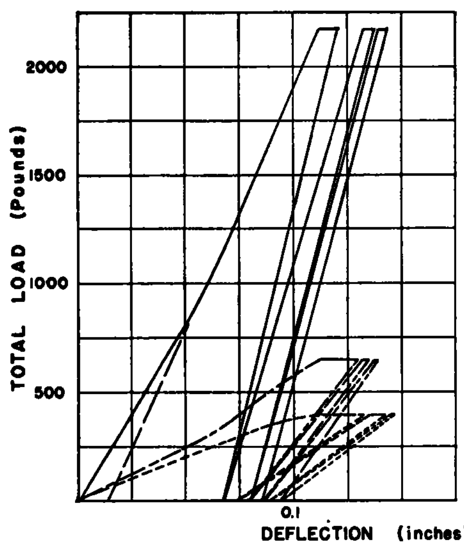

Figure 10. Typical Plot of Total Load Versus Deformation for Data from Plate Tests

ing 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 Figure 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: (1) irregularities of the load-deformation curves; (2) large discrepancies in bearing value from one test with regard to the other two tests; or (3) values causing decreasing stress with increasing perimeter-area ratios. Of the 34 plate tests eliminated, six were 4-in., thirteen were 6-in., four were 9-in. and eleven were 12-in.

The failure of 50 percent of the samples to conform to generally accepted relationships between stress and the perimeter-area ratio could be attributed to the following factors: (1) working in a plate-size range of recognized questionability; (2) irregularities in material tested; (3) unfavorable test conditions, such as proximity of dial stand and end posts to the loaded area, and minor irregularities in rate of loading; and (or) (4) human errors.

For the purposes of an empirical analysis, the bearing capacities plotted in Figure 12 for 12-in. 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-in. diameter plate.

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

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

Base Samples—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 PI of the material passing the No. 40 sieve should be no greater than 6.

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

In Table 8, there is a list of the base samples that had a PI 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 Figures 7, 8, 9, 12 and 13. Nothing of significance was noted. However, at only nine locations was the mat thickness within ± 3 in. 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 9, 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

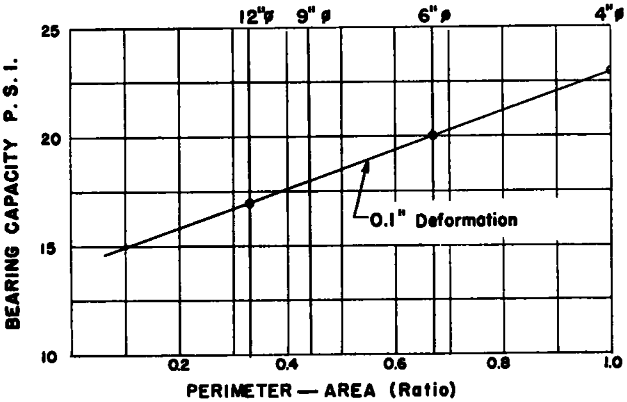

Figure 11. Plot of Bearing Capacity 0.1-in. Deformation, Single Loading Versus Perimeter-Area Ratio

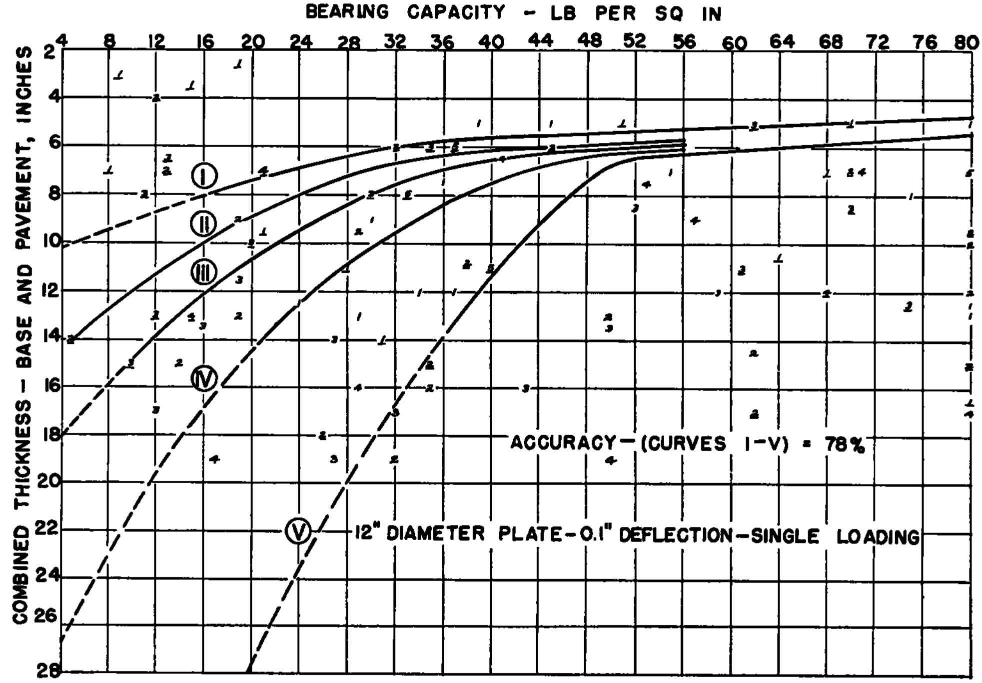

Figure 12. Plot of Bearing Capacity for 12-in. Diameter Plate, 0.1-in. Deflection—Single Loading Versus Combined Thickness Above Subgrade—Traffic and performance are plotted as in Fig. 7.

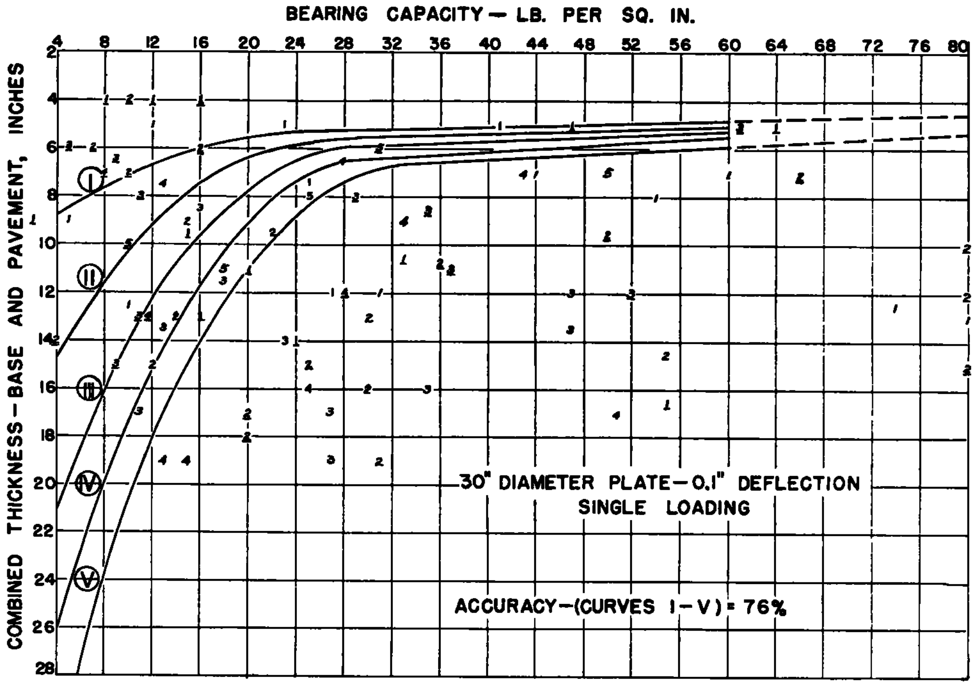

Figure 13. Plot of Bearing Capacity for 30-in. Diameter Plate, 0.1-in. Deflection, Single Loading Versus Combined Thickness Above Subgrade—Traffic and performance are plotted the same as in Fig. 7.

checked against the five sets of curves, but again there was no significant trend noted. Only 15 had a mat thickness within 3 in. 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.

TABLE 8
LIST OF BASE SAMPLES WITH PI GREATER THAN SIX
(Material Passing No. 40 Sieve)

Location Number	Pavement Performance	Mat Thickness	PI	Percent Passing No. 200 Sieve	Percent Passing No. 40 Sieve
		in.			
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

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

Relationship between Test Results—Numerous investigators have attempted to estimate the relationships between the several proposed methods for evaluating subgrade 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 Figure 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.

TABLE 9
LIST OF BASE SAMPLES WITH MORE THAN 8 PERCENT PASSING THE NO. 200 SIEVE

Location Number	Pavement Performance	Mat Thickness	Percent Passing No. 200	PI (Material Passing No. 40 Sieve)
		in.		
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

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 modification.

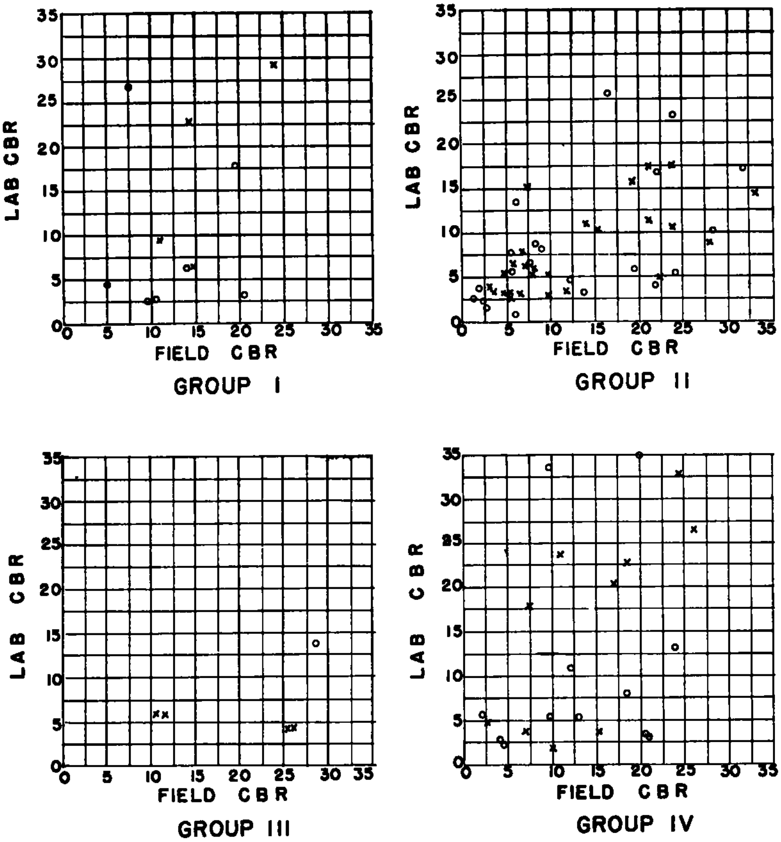

Figure 14. The Relationship between Laboratory CBR and Field CBR

Group I **Samples Whose Densities are within Plus or Minus Three Pounds Per Cubic Foot and Moisture Contents within Plus or Minus Two Percent**
Group II **Samples Whose Moisture Contents are within Plus or Minus Two Percent**
Group III **Samples Whose Densities are within Plus or Minus Three Pounds Per Cubic Foot**
Group IV **Samples Not Included in Previous Groups**

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 Figure 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-in. deformation, and single loading (Figure 16).

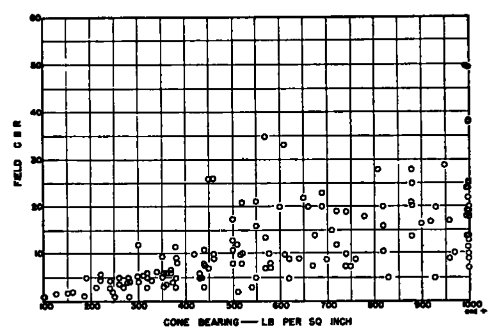

Figure 15. Minimum Field CBR Value Plotted against Average North Dakota Cone Bearing Value for Same Location

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-in. plate, 0.1-in. deformation, single loading, shown in Figure 17. Even so, there is considerable variation throughout the range of bearing values.

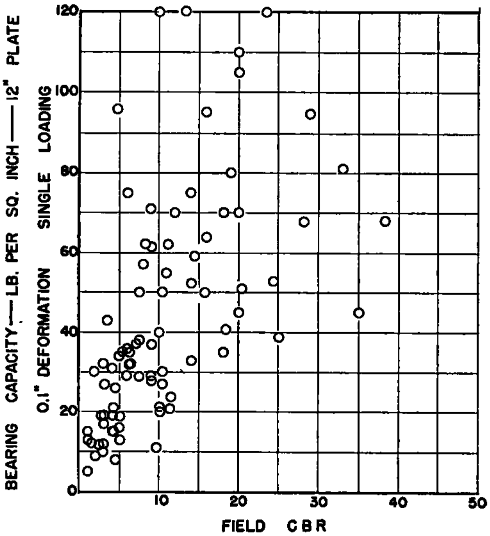

Figure 16. Minimum Field CBR Value Plotted against Bearing Capacity for 12-in. Diameter Plate, 0.1-in. Deformation, Single Loading

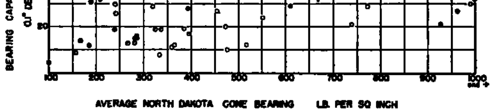

Figure 17. Plot of North Dakota Cone Bearing Value Versus Bearing Capacity for 12-in. Diameter Plate, 0.1-in. Deformation, Single Loading

RESULTS

Inasmuch as several influencing factors have not been taken into account in some phases of the analysis—particularly in development of thickness versus bearing value curves—the results of this work are not considered conclusive for design at this stage.

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 over all accuracy of from 77 to 83 percent, where as 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 (ASTM 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 comparative value between the two tests.

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. Modifications for peculiar circumstances may increase this somewhat, particularly for laboratory tests where conditions of moisture and density were far different from those existing in the field. Even so it is probable that the accuracy would be changed only a small percentage at best. The influence of construction techniques—particularly stage construction over a long period of time—cannot be definitely determined in a study of this type. Adverse seasonal conditions plus concurrent heavy traffic sometimes produce rapid deterioration that cannot be properly evaluated unless bearing tests are made in the field at the time of deterioration.

To make bearing tests adaptable for design purposes, it is necessary that some correlation between a laboratory test and field performance be established.

Although a dependable correlation between field and laboratory tests on subgrades has not been established thus far in these data, it is possible that additional data and further analysis of existing information will produce a more definite relationship.

ACKNOWLEDGMENTS

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; and Messrs. J. A. Bitterman, Engineer of Tests and O. R. Threlkeld, Soils Engineer in the Testing Laboratory. 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.

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 on 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.* Highway Research Board.
20. McLeod, N. W., "Airport Runway Evaluation in Canada," Highway Research Board, August 1947.
21. Goldbeck, A. T., "The Problem of Flexible Pavement Design," *The Crushed Stone Journal*, June 1948.

CURRENT DESIGN OF CONCRETE PAVEMENT IN NEW JERSEY

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SYNOPSIS

This paper pertains to the design of concrete pavement currently employed in New Jersey in the construction of highways that are to carry heavy truck traffic. The design features described and discussed are: (1) pavement thickness; (2) slab length and width; (3) reinforcing steel; (4) dowelling of expansion joints; (5) dowel coatings; (6) joint filler; and (7) sub-base construction. Information is given in regard to the performance of certain expansion joints employed in the past. Discussed are: (1) the effects of corrosion on dowels; and (2) the reasons why the design includes expansion joints. Included are: (1) traffic data; (2) the results of load-transfer tests on various sizes, shapes and lengths of dowels; and (3) the general behavior of contraction joint pavement.

The development of a design of concrete pavement that, on a long-term basis, will resist the damaging effects of heavy truck traffic has for a number of years been a problem of major importance in New Jersey. Because of New Jersey's particular geographical location on the Atlantic Seaboard, and in the most highly industrialized area in the country, its primary highways carry an unusually large amount of traffic. A case in point is U. S. Route 1 which, in the vicinity of Newark, carries an average of 56,000 vehicles daily. Of greater significance, however, is the fact that on a number of these primary highways approximately 25 percent of the traffic consists of truck traffic. Furthermore, approximately 10 percent of the total traffic consists of tractor semi-trailer combinations which, in New Jersey, may have a legal gross weight of 60,000 lb. At this legal gross weight, these 3-axle combinations have been found to have an average load of 26,000 lb., on each rear axle. However, axle loads much in excess of 26,000 lb., are common. In conjunction with 13½-in. dual tires an axle load of 34,400 lb., is currently legal.

Traffic volumes and the general make-up of the traffic at various locations on U.S. Route 1 are shown in Table 1. Traffic volumes and axle loadings on several primary highways are shown in Table 2. It should be noted, however, that the data shown in Table 2 do not include locations where traffic is heaviest, because conditions in these locations do not permit loadometer surveys to be made.

In recent years, as observed elsewhere throughout the country in general, both the number and the average weights of trucks have increased materially, and are still on the increase. As indicated in the foregoing, New Jersey's design problem is one that not only concerns high axle loads but also one that concerns a very high frequency of high axle loads.

EFFECT OF TRUCK TRAFFIC

As amply demonstrated on a country-wide basis in recent years, the predominating tendency of heavy truck traffic in passing over a