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Flexible Pavement Design Developments

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Contents

Compaction Requirements for Flexible Pavements

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This paper presents the results of an analytical study made to develop criteria for determining the degree of compaction required at different depths in soils beneath flexible pavements to prevent consolidation of the soil under wheel loads and consequent deformation of the pavement.

Data obtained from observations of airfield pavements in actual service and from reports of accelerated traffic tests on carefully controlled test sections were tabulated, and from these tabulations correlations were developed between the compaction effort applied to flexible pavements by aircraft traffic and the densities resulting from this compaction effort at various depths.

The established CBR relations were used to integrate the effects of wheel load, tire pressure, assembly configuration, and depth below pavement surface into a compaction index, C_i, for purposes of this study. Correlations between C_i and the densities required to prevent further compaction are presented.

IN 1942, when the Corps of Engineers adopted the California Bearing Ratio method r use in designing flexible pavements for airfields, the CBR procedures specified boratory compaction of soil samples under a 2,000-psi static load. This compac- n gives densities of the same order as those obtained by AASHO Method T99 for ndy and gravelly soils, but much higher densities for clayey soils. The CBR meth- also specified a field compaction test using a tamper that imparted a compaction fort considerably greater than imparted by AASHO T99 compaction. Personnel of e Corps of Engineers and consultants to the Corps anticipated that higher densities uld be needed in soil components of airfield pavements than were produced by the SHO T99 compaction test, but did not consider the CBR procedures entirely suit- e for this purpose. From laboratory tests performed in the Corps' Flexible Pave- nt Laboratory, Soils Division, at the Waterways Experiment Station, Vicksburg, ssissippi, it was determined that a modification of AASHO T99 would be better suit- to the Corps' problems and would require less new test equipment. The Corps' de- n manual published in June 1942 specified a laboratory compaction test similar to SHO T99, but with modifications which increased the weight of the hammer from 5 10 lb, the height of fall from 12 to 18 in., and the number of layers compacted from 5. These changes increased the compaction effort almost fivefold.

Also, based primarily on judgment of Corps personnel and consultants, compaction uirements were specified in 1942 as 95 percent of modified AASHO maximum density all base courses, subbases, and for the top 6 in. of subgrades. In most soils 95 per- t of modified AASHO density is equal to or higher than 100 percent of AASHO T99 ximum density; therefore, these specifications represented a definite upgrading of npaction requirements from those used for highways, which were normally 95 per- t of AASHO T99. Compaction of fill was specified to be 90 percent of modified SHO compaction, but no specifications were established for cut sections except in top 6 in.

n 1945, a study was made of the degrees of compaction existing in certain acceler- d-traffic test sections. These studies showed a definite relation between degree of compaction, wheel load, and depth from the surface of the pavement to the layer being studied. It was assumed that if this density had been built into the structure during construction of the test sections, no appreciable densification would have occurred under traffic. As a result of these studies, the Corps has established in a sense, a "design" of the ultimate compaction necessary. For the "design," the compaction that wil be induced in each layer by the airplane traffic is determined, and this degree of compaction is required to be obtained during construction.

Unfortunately, the studies which led to these developments were documented only i letter reports between the Waterways Experiment Station and the Office, Chief of Engineers, and thus the test data have not been generally available. However, in 1959, th Corps published a report (24) which contains all the data collected by the Corps on the subject. The authors of this paper were directly connected with the studies. This paper summarizes data (24) and shows how the procedures developed by the Corps can b applied to civil airfields and highways.

Early Studies

Figure 1, taken from a 1945 unpublished letter report, shows plots of the degree o compaction that developed in several accelerated-traffic tests at various depths below

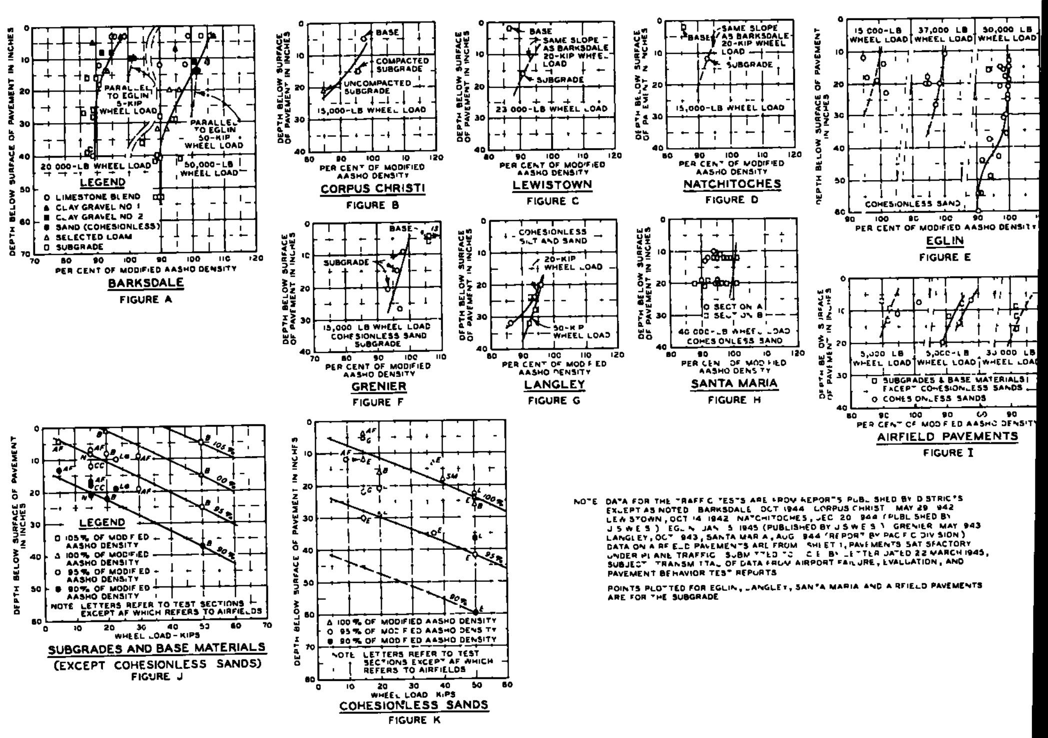
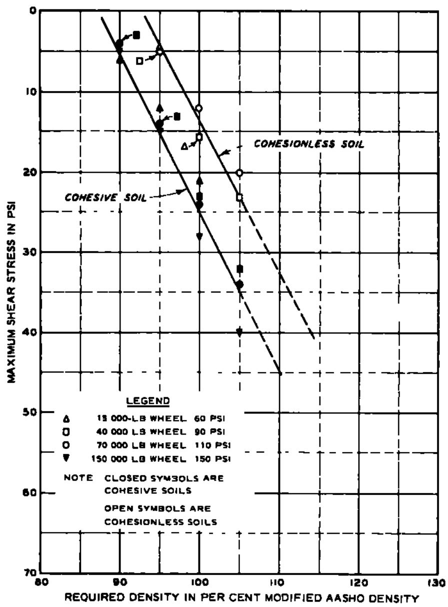


Figure 1. Compaction study data.

the pavement surface. It is apparent that the density developed by traffic decreased with depth in a logical manner when the densities were expressed as a percentage of the maximum densities obtained in the laboratory compaction test. This pattern appeared in all the accelerated-traffic tests (Fig. 1A-H) and in the airfield pavement u der actual traffic (Fig. 2). Another feature indicated by these results is that the co sionless sands appear to plot about 5 points higher (in percentage of compaction) tha the other soils. Figures 1J and 1K are summary plots obtained by reading the dept



igure 2. Required density versus maximum shear stress.

at which 90, 95, 100, and 105 percent compaction were measured and plotting the depth against wheel load. The lines of equal percentage of compaction were fitted to the plotted points visually. These summary plots were used to establish the following compaction requirements which appeared in the Corps of Engineers' engineering design manual published in 1946.

Through the succeeding four years, personnel of the Flexible Pavement Laboratory were engaged in producing CBR versus thickness design curves for multiple-wheel gears and for higher tire pressures by theoretical resolution of the single-wheel curves. These same concepts were applied to the compaction requirements, and it was found that a definite relation existed between the required degree of compaction and the maximum shear stress (τ_{max}) as computed by the theory of elasticity. Figure 2 shows the relation. In 1950, the relation shown in Figure 2 was used to translate the compaction requirements for single wheels (Table 1) into compaction requirements for a range of single, dual, and twin-tandem assemblies. Although tire pressure was not ıdicated in the 1946 requirements, the tire pressures for the various loads were approxiıately those shown in the legend of Figure 2, and these values were used for the translations. ranslations were produced for 100- and 200-psi tire pressures for single-wheel loads. For ıe dual and twin-tandem assemblies, the tire pressure was varied to give a contact area of 57 sq in. for each tire. Figure 3 shows the compaction requirements produced by theoret-

TABLE 1

1946 COMPACTION REQUIREMENTS

heel Load (lb)	Depth in Inches Below Pavement Surface to Which Indicated % of Modified AASHO Density Should Extend: All Subgrades Except Cohesionless Sands		Cohesionless Sands	
	100%	95%	100%	95%
5,000	-	-	-	12
15,000	-	12	12	24
40,000	12	18	24	36
60,000	18	30	30	48
50,000	30	54	48	78

ıl resolution of the 1946 criteria. These requirements appeared in the Corps' engiering design manual in 1951.

In the period following 1951, it was necessary to produce plots such as those shown Figure 3 for many different gear loadings. In the course of this work, ample evince was found that the compaction that will be produced in a given layer by traffic is unction of the total load, arrangement of tires, tire pressure, number of repetitions, ı depth to the given layer. Theoretically, the characteristics of the material between the surface and the given layer should also influence the compaction, but apparently the differences in the materials in the average flexible pavement are not enough to significantly influence compaction.

The determination of the exact relations between the compaction induced in the given layer and each of the variables listed above would require a multiplicity of carefully controlled test sections. A major discovery by personnel of the Flexible Pavement

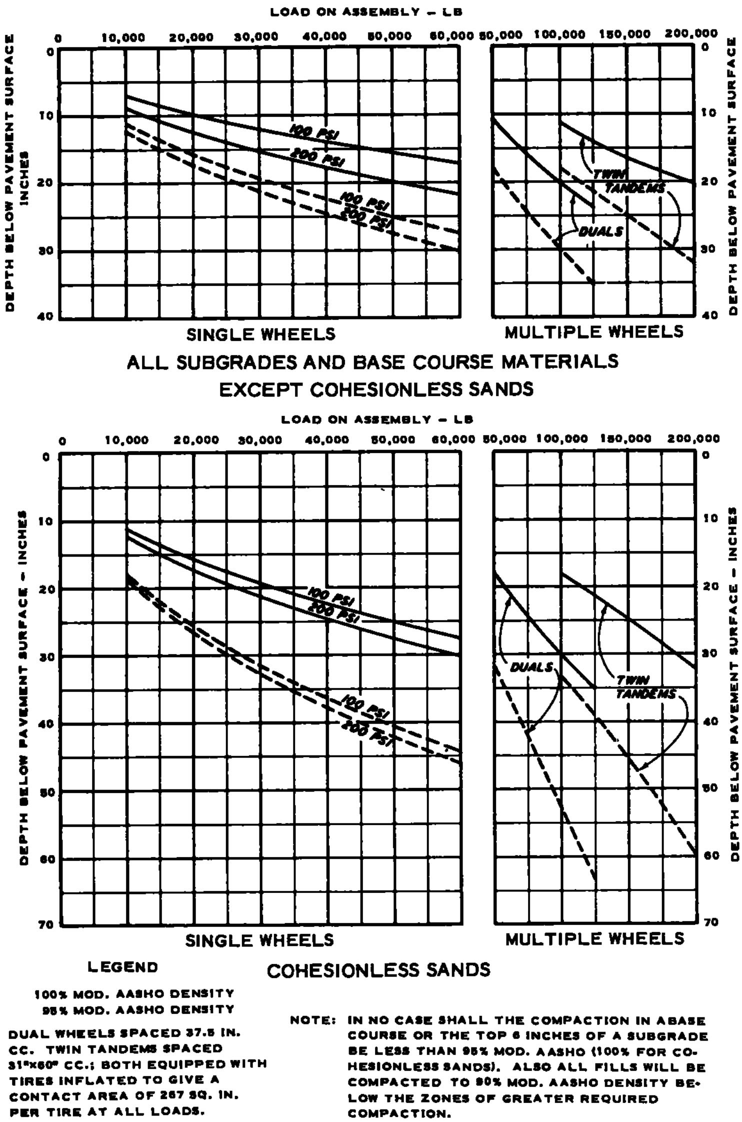


Figure 3. Subgrade and base course compaction requirements.

Laboratory was that the design CBR could be used as a compaction index to combine the parameters listed. In preparing compaction requirements for the various gear loads it was found that an almost constant relation exists between the degree of compaction required in a given layer by the Corps and the design CBR indicated by the Corps' CBR design curves for that layer. Table 2 illustrates the constancy of th relation. The values shown in Table 2 were obtained by selecting a range of loads

TABLE 2

REQUIRED CBR VALUES FOR VARIOUS WHEEL LOADS[1]

	Single Wheels			Multiple Wheels			
ASHO Density	Wheel Load (kips)	CBR for Indicated Tire Pressure 100 psi	200 psi	Assembly Load (kips)	CBR for Dual Wheel Loads	Assembly Load (kips)	CBR for Twin-Tandem Wheel Loads
(a) Cohesionless Sands							
00% Mod.	10	8.1	7.1	50	9.2	100	9.5
	20	8.1	7.2	75	8.6	125	8.9
	30	8.0	7.7	100	8.5	150	9.4
	40	8.0	7.5	125	8.5	175	8.9
	50	8.0	7.4	-	-	200	9.2
	60	8.0	7.5	-	-	-	-
95% Mod.	10	3.7	3.3	50	4.1	100	4.7
	20	3.6	3.4	75	4.0	125	4.6
	30	3.6	3.3	100	3.7	150	4.5
	40	3.5	3.3	125	3.6	175	4.2
	50	3.6	3.3	-	-	200	4.1
	60	3.6	3.6	-	-	-	-
(b) Other Soils							
00% Mod.	10	15	13	50	16	100	16
	20	15.5	13.5	75	14.5	125	15
	30	16	14	100	15	150	15
	40	15.5	14	125	15	175	15.5
	50	16	13.5	-	-	200	16
	60	16	13.5	-	-	-	-
5% Mod.	10	8.1	7.1	50	9.2	100	9.5
	20	8.1	7.2	75	8.6	125	8.9
	30	8.0	7.7	100	8.5	150	9.4
	40	8.0	7.5	125	8.5	175	8.9
	50	8.0	7.4	-	-	200	9.2
	60	8.0	7.5	-	-	-	-

verage CBR: (a) Cohesionless Sands, 100% Mod. AASHO Density = 8.3; 95% = 3.8; (b) her Soils, 100% Mod. AASHO Density = 15.0; 95% = 8.3.

d gear configurations, reading the depth at which 95 and 100 percent compaction uld be required from Figure 3, and then reading from the respective CBR curve the R that would be required at that thickness. For example, Figure 3 indicates that any material other than cohesionless sand, 100 percent compaction would be re- ired at a depth of 7 in. for a 10,000-lb, single-wheel load, 100-psi tire pressure. e Corps' CBR design curves (Fig. 2 of Appendix, (2)) indicate that a design CBR 15 would be required for the 10,000-lb wheel load at a depth of 7 in. The other ues shown in Table 2 were obtained in the same manner. This over-all factor ich combines the parameters of load, tire arrangement, tire pressure, number of etitions, and depth to the layer under consideration was labeled "Compaction Index," to avoid the confusion that would exist if the initials CBR were used. With this nbination factor the variables are reduced to two, percentage of compaction and npaction index, and all pertinent data can be plotted in one plot and brought to bear the problem even though the data from individual tests do not cover the full range he variables.

Following this discovery, a review was made of all available data (4 - 25). Data re considered pertinent only where information was available on the density, depth,

TABLE 3

ACCELERATED TRAFFIC TEST COMPACTION RESULTS

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index*
A. Source of Data: Pavement Mix Design Study for Very Heavy Gear Loads; Pilot Test Section (DRAFT), Jan 1957			
Assembly Load: 240,000 lb			
Assembly Type: Twin tandem, spacing 31 x 60 in., 267-sq-in. contact area			
4.0	NP	104.7	81.0
10.5	NP	105.9	50.4
14.5	NP	105.8	40.0
35.0	28	92.0	13.8
38.0	28	89.2	12.0
58.0	28	83.2	6.6
4.0	NP	106.2	81.0
8.0	NP	103.8	60.5
14.0	NP	104.1	40.5
B. Source of Data: Unpublished data from Columbus AFB test section, 1958			
Assembly Load: 212,000 lb			
Assembly Type: Twin twin, 37-62-37-in. spacing, 267-sq-in. contact area, bicycle-type gear			
4.0	NP	105.0	83.0
12.0	NP	103.6	44.0
17.0	NP	101.0	30.0
21.0	NP	104.8	23.0
25.5	18	106.9	17.5
31.5	P	101.7	13.5
C. Source of Data: Investigation of Effects of Traffic With High Pressure Tires on Asphalt Pavements, TM 3-312, May 1950			
Assembly Load: 30,000 lb			
Assembly Type: Single, 200-psi tire pressure			
16.0	23	100.3	12.5
12.0	23	99.3	19.0
12.0	23	98.1	19.0
12.0	23	98.2	19.0
12.0	23	97.4	19.0
12.0	23	97.0	19.0
Assembly Load: 120,000 lb			
Assembly Type: Twin tandem, 31 x 60 in.			
13.0	23	99.0	22.5
13.0	23	99.0	22.5
13.0	23	100.5	22.5
13.0	23	93.8	22.5
12.0	23	95.3	25.0
D. Source of Data: Investigation of the Design and Control of Asphalt Paving Mixtures, TM 3-254			
Assembly Load: 15,000 lb			
Assembly Type: Single, 50-psi tire pressure			
1.5	7	93.0	50.0
1.5	7	92.0	50.0
3.0	7	98.0	30.0
3.0	7	95.0	30.0
3.0	7	105.0	30.0
5.0	7	93.0	18.5
3.0	NP	103.0	30.0
4.0	NP	98.0	24.0
6.0	NP	96.0	15.0
6.0	NP	97.0	15.0
6.0	NP	97.0	15.0
6.0	NP	99.0	15.0
6.0	NP	96.0	15.0
6.0	NP	96.0	15.0
0.8	NP	108.0	70.0
2.0	P**	94.0	41.0
2.0	P	95.0	41.0
2.0	P	96.0	41.0
2.0	P	98.0	41.0
6.5	7	93.0	14.0
6.5	NP	99.0	14.0
6.5	NP	93.0	14.0
6.5	NP	99.0	14.0
6.0	NP	98.0	14.0
6.0	NP	100.0	15.0
7.0	NP	95.0	13.0
6.5	NP	100.0	14.0
6.5	NP	103.0	14.0
6.5	NP	102.0	14.0
6.5	NP	100.0	14.0
6.5	NP	102.0	14.0
9.0	28	93.0	9.3
9.0	28	96.0	9.3
9.0	28	95.0	9.3
9.0	28	94.0	9.3
9.0	28	95.0	9.3
9.0	28	96.0	9.3
9.0	28	94.0	9.3
9.0	28	94.0	9.3
9.0	28	96.0	9.3
9.0	28	96.0	9.3
9.0	28	95.0	9.3
9.0	28	94.0	9.3
9.0	28	92.0	9.3
9.0	28	95.0	9.3
9.0	28	92.0	9.3
9.0	28	96.0	9.3
11.0	28	94.0	7.0
11.0	28	95.0	7.0
11.0	28	95.0	7.0
D. (Continued)			
Assembly Load: 37,000 lb			
Assembly Type: Single, 100-psi tire pressure			
1.5	7	93.0	98.0
3.0	7	98.0	63.0
3.0	NP	99.0	63.0
5.0	NP	93.0	45.0
2.0	NP	100.0	82.0
6.0	NP	102.0	39.0
2.0	P	79.0	82.0
2.0	P	88.0	82.0
2.0	P	98.0	82.0
2.0	P	94.0	82.0
2.0	P	96.0	82.0
7.25	7	91.0	32.5
8.0	7	89.0	30.0
7.25	NP	101.0	32.5
8.0	NP	101.0	30.0
8.0	NP	101.0	30.0
9.0	NP	91.0	26.0
7.5	NP	101.0	31.5
8.5	NP	100.0	28.0
9.5	NP	102.0	24.0
6.5	P	95.0	35.5
13.0	28	91.0	17.0
13.0	28	90.0	17.0
13.0	28	98.0	17.0
13.0	28	93.0	17.0
13.0	28	97.0	17.0
13.0	28	90.0	17.0
13.0	28	96.0	17.0
13.0	28	97.0	17.0
13.0	28	96.0	17.0
11.0	28	82.0	20.0
11.0	28	91.0	20.0
D. (Continued)			
Assembly Load: 60,000 lb			
Assembly Type: Twin, 37 in. c-c, 360-sq-in. contact area			
3.0	7	87.0	61.0
3.0	7	89.0	61.0
3.0	NP	100.0	61.0
2.0	P	93.0	75.0
2.0	P	94.0	75.0
2.0	P	94.0	75.0
2.0	P	83.0	75.0
2.0	P	94.0	75.0
8.75	7	95.0	24.0
9.5	7	89.0	21.0
9.5	7	79.0	21.0
8.75	NP	98.0	24.0
8.75	NP	102.0	24.0
8.75	NP	95.0	24.0
9.5	NP	97.0	21.0
10.5	NP	101.0	19.0
9.0	NP	101.0	23.0
9.0	NP	101.0	23.0
9.0	NP	97.0	23.0
10.0	NP	100.0	20.0
11.0	NP	99.0	18.0
11.0	NP	99.0	18.0
4.9	NP	103.0	43.0
16.0	28	97.0	12.5
16.0	28	93.0	12.5
16.0	28	95.0	12.5
16.0	28	91.0	12.5
16.0	28	96.0	12.5
16.0	28	93.0	12.5
16.0	28	88.0	12.5
16.0	28	96.0	12.5
16.0	28	96.0	12.5
16.0	28	88.0	12.5
16.0	28	95.0	12.5
16.0	28	96.0	12.5
16.0	28	93.0	12.5
16.0	28	92.0	12.5
16.0	28	92.0	12.5
16.2	28	95.0	12.3
11.0	28	96.0	18.0
11.0	28	93.0	18.0

FIELD COMPACTION DATA FOR FLEXIBLE AIRFIELD PAVEMENTS

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index*
A. Source of Data: Condition Survey, Report No. 2, Pope Air Force Base, Fort Bragg, North Carolina, MP 4-3			
Assembly Load: 13,000 lb			
Assembly Type: Single, 100-psi tire pressure			
4.0	NP	95.0	37.0
3.0	NP	93.0	48.0
9.0	7	83.0	13.5
21.0	13	84.0	3.4
20.0	NP	85.0	3.8
8.0	NP	81.0	15.0
B. Source of Data. Condition Survey, Report No. 5, Eglin Air Force Base, Valparaiso, Florida, MP 4-3			
Assembly Load: 30,000 lb			
Assembly Type. Single, 100-psi tire pressure			
8.0	NP	101.5	27.0
15.0	NP	96.9	11.5
4.0	NP	97.2	52.0
16.0	NP	94.9	10.5
8.0	NP	98.2	27.0
Assembly Load: 96,000 lb			
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area			
20.0	NP	102.7	15.5
15.0	NP	98.5	22.5
24.0	NP	96.7	12.0
36.0	NP	92.0	6.8
C. Source of Data: Airfield Pavement Evaluation, Report No. 3, Boca Raton Airfield, Florida, TM 3-344			
Assembly Load: 62,000 lb			
Assembly Type: Dual, 37 in. c-c, 360-sq-in. contact area			
11.0	NP	96.0	17.8
25.5	NP	96.0	6.8
10.5	NP	94.0	18.5
24.0	NP	84.0	7.5
10.75	NP	91.0	18.1
10.0	NP	96.0	20.7
24.0	NP	91.0	7.5
11.0	NP	96.0	17.8
24.0	NP	93.0	7.5
D. Source of Data: Condition Survey, Report No. 3, Lawson Air Force Base, Fort Benning, Georgia, MP 4-3			
Assembly Load: 13,000 lb			
Assembly Type: Single, 100-psi tire pressure			
3.0	6	89.0	48.0
3.0	11	89.0	48.0
3.0	NP	89.0	48.0
3.0	NP	89.0	48.0
3.0	NP	90.0	48.0
3.0	NP	89.0	48.0
3.0	NP	89.0	48.0
11.0	NP	89.0	9.75
11.0	NP	88.0	9.75
12.0	NP	93.0	8.5
12.0	NP	92.0	8.5
10.0	NP	90.0	11.0
10.0	NP	85.0	11.0
12.0	NP	85.0	8.5
E. Source of Data: Condition Survey, Report No. 4, Ardmore Air Force Base, Ardmore, Oklahoma, MP 4-3			
Assembly Load: 22,500 lb			
Assembly Type: Single, 100-psi tire pressure			
3.0	10	102.0	57.0
3.0	6	98.0	57.0
16.0	11	92.0	8.5
14.0	8	89.0	10.0
F. Source of Data. Airfield Pavement Evaluation, Report No. 6, Palm Beach International Airport, Florida, TM 3-344			
Assembly Load: 79,000 lb			
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area			
3.0	NP	99.0	81.0
3.0	NP	100.0	81.0
7.0	NP	95.0	42.0
4.75	NP	93.0	60.0
2.0	NP	94.0	96.0
4.5	NP	95.0	62.0
3.5	NP	98.0	75.0
3.0	NP	101.0	80.0
3.0	NP	103.0	80.0
3.25	NP	104.0	78.0
4.0	NP	100.0	69.0
6.0	NP	103.0	49.0
3.5	NP	99.0	75.0
13.0	NP	89.0	21.0
13.5	NP	92.0	20.0
13.0	NP	92.0	21.0
13.0	NP	90.0	21.0
13.0	NP	91.0	21.0
11.5	NP	92.0	23.0
F. (Continued)			
21.0	NP	102.0	12.0
33.0	NP	95.0	6.4
14.5	NP	92.0	18.5
26.5	NP	95.0	8.8
17.5	NP	89.0	15.0
29.5	NP	90.0	7.5
8.0	NP	82.0	36.0
20.0	NP	81.0	13.0
11.0	NP	93.0	24.5
23.0	NP	91.0	11.0
14.0	NP	97.0	19.0
26.0	NP	94.0	9.0
19.0	NP	95.0	13.5
31.0	NP	94.0	6.6
11.5	NP	98.0	23.0
23.5	NP	97.0	10.5
19.0	NP	95.0	13.5
31.0	NP	95.0	6.6
17.0	NP	93.0	15.5
29.0	NP	95.0	7.75
17.0	NP	100.0	15.5
29.0	NP	101.0	7.75
14.5	NP	97.0	18.5
26.5	NP	97.0	8.8
16.5	NP	98.0	16.0
28.5	NP	90.0	8.0
12.0	NP	95.0	22.5
24.0	NP	94.0	10.0
13.5	NP	101.0	20.0
25.5	NP	94.0	9.4
G. Source of Data: Airfield Pavement Evaluation, Report No. 2, Sheppard Air Force Base, Wichita Falls, Texas, TM 3-344			
Assembly Load. 15,750 lb			
Assembly Type: Single, 100-psi tire pressure			
3.0	8	100.0	51.0
12.0	NP	87.0	9.8
3.0	NP	94.0	51.0
2.0	7	100.0	66.0
9.0	NP	89.0	14.5
2.5	NP	90.0	58.0
3.0	NP	97.0	51.0
20.0	7	79.0	4.3
14.5	11	94.0	7.4
23.0	18	89.0	3.3
13.0	18	85.0	8.8
15.0	28	89.0	7.0
2.5	NP	91.0	58.0
3.0	NP	87.0	51.0
2.5	NP	93.0	58.0
12.0	17	93.0	9.8
13.5	20	91.0	8.2
14.0	17	93.0	7.7
H. Source of Data: Airfield Pavement Evaluation, Report No. 4, Davis-Monthan Air Force Base, Tuscon, Arizona, TM 3-344			
Assembly Load: 74,400 lb			
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area			
3.5	15	94.7	71.0
3.5	4	99.7	71.0
4.0	10	98.5	65.0
4.0	18	97.3	65.0
2.5	NP	97.9	85.0
4.0	NP	98.4	65.0
3.5	11	100.7	71.0
11.5	20	86.3	22.5
11.5	11	92.7	22.5
12.0	17	92.2	21.0
11.0	19	96.9	23.5
11.5	23	88.3	22.5
12.0	12	90.2	21.0
14.0	12	89.6	18.0
12.0	13	90.3	21.0
12.0	12	91.4	21.0
12.0	8	97.6	21.0
14.0	13	95.5	18.0
I. Source of Data: Flexible Pavement Behavior Studies, Interim Report No. 2			
Assembly Load: 15,000 lb			
Assembly Type: Single, 100-psi tire pressure			
15.0	NP	92.0	6.75
15.0	NP	94.0	6.75
15.0	NP	95.0	6.75
15.0	7	94.0	6.75
20.0	44	86.0	4.0
20.0	44	92.0	4.0
15.0	37	87.0	6.75
15.0	11	84.0	6.75
15.0	13	94.0	6.75
19.0	32	86.0	4.5
12.0	13	92.0	9.5
13.0	1	86.0	8.5
17.0	8	75.0	5.5
13.0	1	94.0	8.5
13.0	6	94.0	8.5
Assembly Load: 16,000 lb			
Assembly Type: Single, 100-psi tire pressure			
11.0	17	79.0	12.0
11.0	16	91.0	12.0
19.0	20	69.0	4.8
24.0	20	73.0	3.2
19.0	18	74.0	4.8
24.0	18	72.0	3.2

(Continued)

* The compaction index is the design CBR value for the corresponding load and depth.

TABLE 4 (Continued)

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
I. (Continued)			
Assembly Load:	17,500 lb		
Assembly Type:	Single, 100-psi tire pressure		
16.0	20	85.0	6.8
13.0	11	88.0	9.5
14.0	16	84.0	8.5
25.0	17	69.0	3.2
22.0	10	79.0	3.9
25.0	15	78.0	3.2
Assembly Load:	25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
15.0	20	87.0	10.0
24.0	20	79.0	4.7
15.0	20	90.0	10.0
24.0	20	83.0	4.7
20.0	9	83.0	6.4
26.0	9	85.0	4.0
17.0	2	90.0	8.3
13.0	9	106.0	12.3
15.0	14	94.0	10.0
15.0	7	84.0	10.0
15.0	7	104.0	10.0
14.0	15	75.0	11.0
15.0	5	75.0	10.0
J. Source of Data:	Airfield Pavement Evaluation, Report No. 1, Campbell Air Force Base, Kentucky, TM 3-344		
Assembly Load:	140,000 lb		
Assembly Type.	Twin tandem, 31 x 60 in. c-c, 267-sq-in. contact area		
12.5	29	89.0	22.5
24.5	P	72.0	11.0
32.0	36	91.0	7.75
40.0	P*	88.0	5.6
13.0	P	87.0	21.4
25.0	P	80.0	10.8
11.5	P	89.0	24.0
23.5	P	84.0	11.7
13.5	P	94.0	20.5
25.5	P	82.0	10.6
13.5	P	90.0	20.5
25.5	P	83.0	10.6
30.0	P	88.0	8.4
42.0	P	88.0	5.3
28.0	34	91.0	9.2
40.0	P	88.0	5.6
10.5	26	92.0	25.5
22.5	P	88.0	12.2
31.5	18	91.0	7.9
14.5	P	90.0	19.2
26.5	P	87.0	9.8
30.0	P	95.0	8.4
42.0	P	90.0	5.3
3.0	NP	89.0	98.0
3.5	NP	100.0	90.0
14.0	NP	96.0	20.0
4.5	NP	105.0	55.0
J. (Continued)			
3.0	NP	102.0	98.0
3.0	NP	98.0	98.0
3.0	NP	99.0	98.0
3.5	NP	99.0	90.0
12.5	NP	94.0	22.5
3.5	NP	98.0	90.0
13.0	NP	93.0	21.4
18.5	NP	96.0	15.5
3.5	NP	110.0	90.0
3.5	NP	93.0	90.0
14.0	NP	100.0	20.0
21.0	NP	94.0	13.5
2.5	NP	109.0	81.0
4.5	NP	101.0	55.0
14.0	NP	99.0	20.0
K. Source of Data:	Field Moisture Content Investigation Unpublished Data		
Field:	Ardmore Air Force Base		
Facility:	NS runway		
Assembly Load:	22,000 lb		
Assembly Type	Single, 100-psi tire pressure		
5.5	10	102.0	33.0
5.5	6	98.0	33.0
19.0	CL	89.0	6.25
Field:	Bergstrom Air Force Base		
Facility:	NW-SE runway		
Assembly Load·	15,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
4.5	1	104.0	34.0
4.5	1	101.0	34.0
14.5	NP	92.0	7.0
19.5	44	86.0	4.25
24.0	44	85.0	3.0
4.5	1	101.0	34.0
14.5	NP	90.0	7.0
19.5	44	86.0	4.25
24.0	44	84.0	3.0
4.5	1	104.0	34.0
14.5	NP	89.0	7.0
19.5	44	94.0	4.25
24.0	44	94.0	3.0
4.5	1	104.0	34.0
14.5	NP	94.0	7.0
19.5	44	92.0	4.25
24.0	44	91.0	3.0
4.5	1	104.0	34.0
19.5	44	94.0	4.25
24.0	44	86.0	3.0
4.5	1	97.0	34.0
14.5	NP	95.0	7.0
19.5	44	92.0	4.25
24.0	44	92.0	3.0
K. (Continued)			
Facility:	Taxiway 1		
Assembly Load·	15,000 lb		
Assembly Type·	Single, 100-psi tire pressure		
4.5	3	104.0	34.0
14.5	7	94.0	7.0
19.5	39	86.0	4.25
24.0	39	85.0	3.0
4.5	2	100.0	34.0
14.5	8	91.0	7.0
19.5	38	84.0	4.25
24.0	38	82.0	3.0
Facility.	E-W runway		
Assembly Load:	15,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
4.5	1	102.0	34.0
14.5	7	89.0	7.0
19.5	31	91.0	4.25
4.5	1	102.0	34.0
14.5	8	93.0	7.0
19.5	53	99.0	4.25
4.5	NP	108.0	34.0
14.5	4	90.0	7.0
19.5	38	94.0	4.25
Field.	Berry Air Force Base		
Facility:	West N-S runway		
Assembly Load:	15,000 lb		
5.5	2	102.0	27.5
14.5	37	87.0	7.0
24.0	37	69.0	3.0
5.5	2	106.0	27.5
14.5	11	84.0	7.0
24.0	11	85.0	3.0
5.5	2	108.0	27.5
14.5	20	82.0	7.0
24.0	20	89.0	3.0
5.5	2	109.0	27.5
14.5	29	91.0	7.0
24.0	29	90.0	3.0
Field	Blythe Air Force Base		
Facility:	N-S runway		
Assembly Load:	25,000 lb		
4.5	NP	98.0	43.0
10.5	NP	88.0	16.0
24.0	NP	82.0	4.6
4.5	NP	101.0	43.0
10.5	NP	92.0	16.0
24.0	NP	85.0	4.6
3.0	NP	94.0	59.0
6.5	NP	88.0	29.0
24.0	NP	86.0	4.6
4.5	NP	103.0	43.0
10.5	NP	92.0	16.0
24.0	NP	92.0	4.6
4.5	NP	100.0	43.0
10.5	NP	94.0	16.0
24.0	NP	88.0	4.6
K. (Continued)			
Field:	Campbell Air Force Base		
Facility	N-S runway		
Assembly Load	25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
6.25	NP	102.0	31.0
14.25	20	87.0	11.0
24.0	20	79.0	4.6
5.5	NP	101.0	35.0
14.5	20	90.0	10.6
24.0	20	83.0	4.6
5.5	NP	103.0	35.0
14.5	20	96.0	10.6
24.0	20	95.0	4.6
5.5	NP	106.0	35.0
14.5	20	87.0	10.6
24.0	20	91.0	4.6
10.0	20	90.0	17.0
Facility:	NE-SW runway		
Assembly Load:	15,000 lb		
Assembly Type·	Single, 100-psi tire pressure		
6.0	NP	99.0	25.0
15.5	15	88.0	6.4
Field:	Clovis Air Force Base		
Facility:	N-S runway		
Assembly Load	30,000 lb		
Assembly Type	Single, 100-psi tire pressure		
4.0	7	100.0	52.0
16.0	9	98.0	10.5
32.0	9	86.0	3.3
4.0	7	98.0	52.0
16.0	9	103.0	10.5
33.0	9	84.0	3.0
4.0	7	98.0	52.0
16.0	9	76.0	10.5
32.0	9	102.0	3.3
Field.	Davis Air Force Base		
Facility·	E-W runway		
Assembly Load·	65,000 to 75,000 lb		
Assembly Type:	Dual, 37 in. c-c, 267-sq-in. contact area		
6.0	15	95.0	45.0
14.0	20	86.0	18.5
23.0	20	92.0	10.25
6.0	4	100.0	45.0
14.0	11	93.0	18.5
24.0	11	76.0	9.6
Facility:	N-S runway		
Assembly Load	65,000 to 75,000 lb		
Assembly Type	Dual, 37 in. c-c, 267-sq-in. contact area		
6.5	10	99.0	42.5
14.5	17	92.0	17.75
24.0	17	89.0	9.6

K. (Continued)

Facility: Taxiway 4
Assembly Load: 65,000 to 75,000 lb
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	18	97.0	42.5
13.5	19	96.0	19.0
24.0	19	81.0	9.6

Facility: Taxiway 3
Assembly Load: 65,000 to 75,000 lb
Assembly Type: Dual, 37 in c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.0	NP	100.0	45.0
14.0	23	88.0	18.5
24.0	23	83.0	9.6

Facility: Taxiway 2
Assembly Load: 65,000 to 75,000 lb
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	10	98.0	42.5
14.5	12	90.0	17.75
23.0	12	76.0	10.25

Facility: Taxiway 1
Assembly Load: 65,000 to 75,000 lb
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	10	95.0	42.5
16.5	12	90.0	15.0
24.0	12	77.0	9.6

Facility: Taxiway 9
Assembly Load. 65,000 to 75,000 lb
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.0	10	91.0	45.0
14.5	13	90.0	17.75
26.0	13	93.0	9.5

Facility: NW-SE runway
Assembly Load: 65,000 to 75,000 lb
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
14.5	12	91.0	17.75
23.0	12	82.0	10.25
6.5	NP	102.0	42.5
14.5	8	98.0	17.75
24.0	8	76.0	9.6

Facility: N-S runway
Assembly Load: 65,000 to 75,000 lb
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.0	11	98.0	45.0
14.0	13	95.0	18.5
24.0	13	95.0	9.6

K. (Continued)

Field: Dodge City Air Force Base
Facility: Taxiway 4A
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	9	94.0	22.5
15.5	17	87.0	6.3
22.5	26	82.0	3.3
24.0	26	74.0	3.0
6.5	11	86.0	22.5
15.5	13	94.0	6.3
22.5	32	86.0	3.3
24.0	32	78.0	3.0

Field: Douglas Air Force Base
Facility: Taxiway 5
Assembly Load: 17,500 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	3	93.0	29.0
14.0	NP	87.0	8.5
23.0	39	82.0	3.6
5.5	3	94.0	29.0
14.0	NP	86.0	8.5
23.0	36	81.0	3.6

Facility. N-S runway
Assembly Load: 17,500 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	97.0	29.0
13.0	3	97.0	9.5
22.0	21	86.0	4.0
5.5	NP	94.0	29.0
11.0	3	89.0	12.0
21.0	21	89.0	4.3
5.5	NP	103.0	29.0
14.5	3	92.0	8.0
21.5	15	85.0	4.1
5.5	NP	97.0	29.0
14.5	3	85.0	8.0
21.5	21	88.0	4.1
5.5	9	101.0	29.0
13.5	11	95.0	9.0
21.5	11	85.0	4.1
5.5	9	102.0	29.0
11.5	11	91.0	11.3
17.5	24	80.0	5.9
5.5	9	98.0	29.0
11.5	11	94.0	11.3
16.5	24	87.0	6.5
5.5	9	98.0	29.0
10.5	11	91.0	12.8
18.5	24	84.0	5.4

K. (Continued)

Field: Gainesville Air Force Base
Facility: N-S runway
Assembly Load: 25,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	9	107.0	43.0
16.5	29	92.0	8.8
4.5	13	104.0	43.0
14.5	20	85.0	10.6
28.5	22	89.0	3.4
4.5	8	103.0	43.0
14.5	21	90.0	10.6
24.5	20	82.0	4.5

Field. Jackson Air Force Base
Facility: NW-SE runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	8	104.0	34.0
11.5	13	86.0	10.0
24.0	13	89.0	3.0
4.5	8	103.0	34.0
11.5	13	92.0	10.0
24.0	13	90.0	3.0

Field: Keesler Air Force Base
Facility. NW-SE runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	NP	104.0	34.0
4.5	NP	103.0	34.0
4.5	NP	98.0	34.0
11.5	NP	101.0	10.0
4.5	NP	103.0	34.0
13.5	NP	99.0	8.0

Field. Kirtland Air Force Base
Facility. Taxiway 2
Assembly Load. 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.0	5	106.0	38.3
13.0	5	95.0	8.5
4.0	4	102.0	38.3
13.0	6	94.0	8.5
4.0	3	108.0	38.3
13.0	4	93.0	8.5

Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	3	99.0	43.0
13.5	NP	90.0	13.3
24.0	NP	84.0	5.5
4.5	4	91.0	47.0
13.5	8	82.0	13.3
24.0	8	86.0	5.5

K. (Continued)

Facility: N-S runway
Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	6	98.0	47.0
12.5	4	90.0	14.5
24.0	4	72.0	5.5
5.0	6	95.0	43.0
13.5	4	92.0	13.3
24.0	4	75.0	5.5
5.0	6	94.0	43.0
13.5	4	83.0	13.3
24.0		77.0	5.5
5.0	5	104.0	43.0
14.0	NP	95.0	12.6
5.0	5	99.0	43.0
11.5	NP	96.0	16.2
24.0	NP	85.0	5.5

Facility: NE-SW runway
Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	NP	104.0	47.0
14.5	3	74.0	12.0
24.0	3	75.0	5.5
4.5	NP	108.0	47.0
16.5	2	87.0	10.0
24.0	2	84.0	5.5
5.0	NP	103.0	43.0
16.5	3	90.0	10.0
24.0	3	76.0	5.5
4.5	NP	102.0	47.0
16.5	2	89.0	10.0
24.0	2	77.0	5.5

Facility: Taxiway 2
Assembly Load: 75,000 lb
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
13.5	4	91.0	19.0
5.0	SC*	96.0	55.0
14.0	SC	86.0	18.5
5.0	SC	96.0	55.0
12.5	NP	94.0	20.5
5.0	SC-SM	104.0	55.0
12.5	SC-SM	90.0	20.5
5.0	SC-SM	97.0	55.0
12.5	SC-SM	92.0	20.5
5.0	SC	102.0	55.0
5.0	SC-SM	103.0	55.0
12.5	NP	94.0	20.5
5.0	SP-SM	100.0	55.0
13.5	NP	94.0	19.0

Facility: Taxiway 1
Assembly Load: 75,000 lb
Assembly Type. Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	8	99.0	59.0
4.5	NP	99.0	59.0

(Continued)

* Classification given where Atterberg limits are unknown.

TABLE 4 (Continued)

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
K. (Continued)			
Field	La Junta Air Force Base		
Facility:	E-W runway		
Assembly Load:	17,500 lb		
Assembly Type	Single, 100-psi tire pressure		
9.5	9	104.0	14.7
15.5	20	85.0	7 2
24.5	17	69.0	3.2
9.5	9	108.0	14.7
15.5	20	85.0	7.2
24 5	17	84.0	3.2
7.0	14	100.0	22.0
14.0	10	91.0	8.5
7.0	14	102 0	22.0
13.0	11	89.0	9.5
22.0	11	79.0	3.9
9 5	8	98.0	14.7
15.5	10	89.0	7.2
Facility	Taxiway 5		
Assembly Load	17,500 lb		
Assembly Type.	Single, 100-psi tire pressure		
7.5	4	100.0	20.0
13.5	16	84.0	9.0
24.5	15	78.0	3.2
7.5	3	100.0	20.0
13.5	16	75.0	9.0
24.5	15	69.0	3.2
Field	Las Vegas Air Force Base		
Facility.	Taxiway 3		
Assembly Load	30,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
7.0	6	100.0	30.0
15.5	16	82 0	11.0
24.0	16	73.0	5.5
6.5	11	109.0	33.0
15.5	8	76.0	11.0
24.0	8	80.0	5.5
Facility.	N-S runway		
Assembly Load	30,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
6.5	NP	103.0	33.0
14.0	26	91.0	12.6
18.5	17	75.0	8.4
24.0	17	79.0	5.5
6.5	8	100.0	33.0
14.0	10	90.0	12.6
18.5	17	90.0	8.4
6.5	5	101.0	33.0
14.0	12	88.0	12.8
18.5	17	91.0	8.4
6.5	5	101.0	33.0
14.0	17	88.0	12.8
18.5	17	85.0	8.4
6.5	3	100.0	33.0
14.0	NP	89.0	12.6
18.5	8	86.0	8.4
K. (Continued)			
6.5	6	103.0	33.0
14.0	18	88.0	12.6
18.5	17	92.0	8.4
24.0	17	83.0	5.5
6.5	5	100.0	33.0
14.0	3	89.0	12.6
18.5	8	79.0	8.4
24.0	8	82.0	5.5
Facility	NE-SW runway		
Assembly Load:	30,000 lb		
Assembly Type	Single, 100-psi tire pressure		
6.5	8	100.0	33.0
13.5	NP	98.0	13.3
21.0	NP	89.0	5.9
6.5	6	96.0	33.0
13.5	11	91.0	13.3
24.0	11	83.0	5.5
6.5	5	97.0	33.0
13.5	1	93.0	13.3
20.0	1	99.0	7.4
6.5	8	99.0	33.0
13.5	NP	95.0	13.3
Field:	Lawson Air Force Base		
Facility.	Taxiway 6		
Assembly Load	15,000 lb		
Assembly Type	Single, 100-psi tire pressure		
4.5	3	84.0	34.0
12.0	1	86.0	9.5
14.0	1	75.0	7.5
4.5	3	88.0	34.0
12.5	1	94.0	9.0
14.0	1	77.0	7.5
Facility	Taxiway 4		
Assembly Load	15,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
5.5	5	87.0	27.5
12.5	20	75.0	9.0
5.5	5	87.0	27.5
13.5	20	89.0	8.0
23.0	20	89.0	3.2
Facility	NE-SW runway		
Assembly Load:	15,000 lb		
Assembly Type	Single, 100-psi tire pressure		
4.5	6	89.0	34.0
12.5	NP	89.0	9.0
4.5	11	89.0	34.0
12.5	NP	88.0	9.0
4.5	NP	89.0	34.0
12.5	NP	93.0	9.0
4.5	NP	89.0	34.0
13.5	NP	92.0	8.0
K. (Continued)			
Facility:	NW-SE runway		
Assembly Load	15,000 lb		
Assembly Type	Single, 100-psi tire pressure		
4.5	NP	90.0	34.0
11.5	NP	90.0	10.0
4.5	NP	89.0	34.0
11.5	NP	85.0	10.0
Facility	NE-SW runway		
Assembly Load:	15,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
4.5	NP	89.0	34.0
11.5	NP	85.0	10.0
Field.	Pope Air Force Base		
Facility:	NE-SW runway		
Assembly Load.	15,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
6.5	NP	95.0	22.5
11.5	7	83.0	10.0
21.0	7	84.0	3.8
5.5	NP	93.0	27.5
10.5	NP	81.0	11.6
20.0	NP	85.0	4.0
Facility	NW-SE runway		
Assembly Load	15,000 to 25,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
5.5	NP	98.0	35.6
15.5	NP	100.0	9.6
23.0	NP	76.0	5.1
6.5	NP	92.0	29.0
14.5	NP	98.0	10.6
Facility:	NE-SW runway		
Assembly Load	15,000 to 25,000 lb		
Assembly Type	Single, 100-psi tire pressure		
7.0	NP	99.0	27.5
16.0	NP	105.0	9.1
25.0	NP	96.0	4.3
7.5	NP	95.0	25.0
13.5	NP	105.0	12.0
23.0	NP	94.0	5.0
7.0	NP	94.0	27.5
16.0	12	89.0	9.1
26.0	12	88.0	4.0
Facility:	N-S runway		
Assembly Load:	15,000 to 25,000 lb		
Assembly Type	Single, 100-psi tire pressure		
7.5	NP	96.0	25.0
12.5	12	91.0	13.1
22.0	12	85.0	5.5
Facility:	Taxiway 1		
Assembly Load	15,000 to 25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
[illegible]	[illegible]	100.0	35.6
K. (Continued)			
Facility	N-S runway		
Assembly Load:	15,000 to 25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
6.0	NP	91.0	31.7
15.0	NP	99.0	10.0
25.0	NP	90.0	4.3
Facility:	Taxiway 1		
Assembly Load:	15,000 to 25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
7.0	NP	102.0	27 5
12.5	NP	97.0	13.1
22.0	NP	98.0	5.5
Facility:	Taxiway 2		
Assembly Load:	15,000 to 25,000 lb		
Assembly Type:	Single, 100-psi tire pressure		
6.5	NP	100.0	29.0
10.5	6	92.0	16.0
20.0	6	77.0	6.4
7.0	NP	100.0	27.5
11.0	NP	109.0	15.1
21.0	NP	99.0	5.9
Facility	Taxiway 5		
Assembly Load	15,000 to 25,000 lb		
Assembly Type	Single, 100-psi tire pressure		
6.0	NP	91.0	31.7
11.0	16	91.0	15.1
21.0	16	81.0	5.9
Facility	Taxiway 1		
Assembly Load.	15,000 to 25,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
6.5	NP	94.0	29.0
15.5	NP	104.0	9 6
25.0	NP	105.0	4.3
Facility	Taxiway 2		
Assembly Load:	15,000 to 25,000 lb		
Assembly Type	Single, 100-psi tire pressure		
6.5	NP	98.0	29.0
13.0	18	98.0	10.0
25.0	18	88.0	4.3
Facility	Taxiway 5		
Assembly Load	15,000 to 25,000 lb		
Assembly Type.	Single, 100-psi tire pressure		
6.0	NP	99.0	31.7
14.0	NP	102.0	11.1
7.0	NP	100.0	27.5
13.5	NP	108.0	12.0

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index

K. (Continued)

Field: Pueblo Air Force Base
Facility: E-W runway
Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	3	102.0	47.0
13.5	24	92.0	13.3
24.0	24	85.0	5.5
4.5	3	99.0	47.0
13.5	20	94.0	13.3
24.0	20	87.0	5.5
4.5	4	98.0	47.0
13.5	20	89.0	13.3
24.0	20	88.0	5.5
4.5	4	95.0	47.0
13.5	20	82.0	13.3
24.0	20	93.0	5.5
4.5	4	102.0	47.0
13.5	20	88.0	13.3
24.0	20	92.0	5.5

Facility: Taxiway 6
Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
7.5	2	100.0	27.9
14.5	9	89.0	12.0
24.0	9	84.0	5.5
7.5	1	97.0	27.9
14.5	18	89.0	12.0
24.0	18	86.0	5.5

Field: Rocky Ford Air Force Base
Facility: E-W runway
Assembly Load: 16,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	94.0	31.5
11.0	17	79.0	11.3
19.0	20	69.0	4.8
24.0	20	73.0	3.0
5.0	5	97.0	31.5
11.0	17	75.0	11.3
19.0	20	71.0	4.8
24.0	20	79.0	3.0
5.0	NP	101.0	31.5
11.0	17	83.0	11.3
19.0	20	77.0	4.8
24.0	20	76.0	3.0
5.0	NP	93.0	31.5
11.0	16	91.0	11.3
19.0	18	74.0	4.8
24.0	18	72.0	3.0
5.0	NP	99.0	31.5
11.0	16	91.0	11.3
19.0	18	74.0	4.8
24.0	18	73.0	3.0
5.0	NP	100.0	31.5
11.0	16	80.0	11.3
19.0	18	70.0	4.8
24.0	18	74.0	3.0

K. (Continued)

Field: Santa Fe Air Force Base
Facility: N-S runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	12	101.0	34.0
12.5	10	78.0	9.0
4.5	11	103.0	34.0
12.5	10	78.0	9.0
4.5	12	104.0	34.0
12.5	17	82.0	9.0

Field: Sewart Air Force Base
Facility: N-S runway
Assembly Load: 25,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
18.5	29	93.0	7.3
24.0	29	91.0	4.6
17.5	24	90.0	8.0
24.0	24	98.0	4.6
14.5	30	86.0	10.6
24.0	30	94.0	4.6
24.0	29	83.0	4.6
17.5	33	89.0	8.0
24.0	33	98.0	4.6
17.5	43	83.0	8.0
24.0	43	72.0	4.6

Facility: Apron
Assembly Load: 45,000 lb
Assembly Type: Dual, 28 in. c-c, 226-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	101.0	33.6
21.5	41	88.0	7.5
29.0	41	90.0	4.8
5.5	NP	94.0	33.6
21.5	34	85.0	7.5
31.0	34	84.0	4.3

Facility: NW-SE runway
Assembly Load: 45,000 lb
Assembly Type: Dual, 28 in. c-c, 226-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	97.0	33.6
18.5	46	90.0	9.0
28.0	46	90.0	5.1
5.5	NP	104.0	33.6
22.5	50	98.0	7.0
28.0	50	92.0	5.1

Facility: New taxiway
Assembly Load: 45,000 lb
Assembly Type: Dual, 28 in. c-c, 226-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	107.0	36.6
21.5	31	90.0	7.5
30.0	31	87.0	4.5
5.0	NP	108.0	36.6
20.0	36	91.0	8.25
30.0	36	96.0	4.5

K. (Continued)

Field: Sheppard Air Force Base
Facility: NE-SW runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	8	100.0	27.5
12.5	NP	87.0	9.0
20.5	7	79.0	3.8

Facility: E-W runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	94.0	31.0
17.0	11	94.0	5.5

Facility: NE-SW runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	7	100.0	34.0
11.5	NP	89.0	10.0

Facility: N-S runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	90.0	31.0
16.5	18	85.0	5.8
5.5	NP	97.0	27.5
17.5	28	89.0	5.2

Facility: Taxiway 5
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	91.0	31.0
14.5	17	93.0	7.0
24.0	17	82.0	3.0

Facility: NW-SE runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	87.0	27.5
16.0	34	91.0	6.0

Facility: E-W runway
Assembly Load: 15,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.0	NP	93.0	31.0
16.5	30	93.0	5.8

Field: South Plains Air Force Base
Facility: NW-SE runway
Assembly Load: 12,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.0	7	92.0	35.0
12.0	13	92.0	9.0
4.0	3	102.0	35.0
12.0	12	98.0	9.0
4.0	NP	105.0	35.0
12.0	13	95.0	9.0

K. (Continued)

Field: West Palm Beach Air Force Base
Facility: NW-SE runway
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 37 in. c-c, 267-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	99.0	63.0
23.5	NP	102.0	12.1
33.0	NP	99.9	7.6

Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	100.0	41.0
16.5	NP	92.0	14.0
26.5	NP	100.2	8.5
9.5	NP	94.0	24.8
20.0	NP	89.0	11.8
29.5	NP	95.5	7.4
6.0	NP	92.0	37.0
12.0	NP	82.0	19.0
20.0	NP	85.9	11.8

Facility: N-S runway
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
4.5	NP	94.0	48.5
13.5	NP	93.0	17.8
23.0	NP	96.5	10.0

Facility: E-W runway
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
7.0	NP	94.0	33.8
17.5	NP	96.0	13.5
26.0	NP	99.0	8.6
6.0	NP	98.0	37.0
21.5	NP	94.0	10.8
31.0	NP	98.9	7.0
14.0	NP	98.0	17.0
23.5	NP	102.7	9.6

Facility: Taxiway A3
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	101.0	41.0
21.5	NP	94.0	10.8
31.0	NP	99.8	7.0

Facility: Taxiway A4
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	103.0	41.0
22.5	NP	93.0	10.0
29.0	NP	99.8	7.6

(Continued)

TABLE 4 (Continued)

K. (Continued)

Facility: Taxiway A3
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	104.0	41.0
19.0	NP	99.0	12.2
29.0	NP	106.5	7.6

Facility: NE-SW runway
Assembly Load. 35,000 to 95,000 lb
Assembly Type· Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	NP	100.0	35.0
8.5	NP	103.0	27.5
17.0	NP	97.0	14.0
26.5	NP	102.7	8.5

Facility· Taxiway A1
Assembly Load· 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.0	NP	99.0	37.0
18.0	NP	97.0	13.3
28.5	NP	95.2	7.8

K. (Continued)

Facility: Apron C
Assembly Load: 35,000 to 95,000 lb
Assembly Type: Dual, 44 in. c-c, 630-sq-in. contact area

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
14.5	NP	94.0	16.5
24.0	NP	99.2	9.5
16.0	NP	100.0	15.0
25.5	NP	99.2	8.9

Field: Woodward Air Force Base
Facility: Taxiway 3
Assembly Load: 25,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
5.5	NP	91.0	35.5
14.5	NP	88.0	10.6
24.0	9	85.0	4.6
5.5	4	95.0	35.0
14.5	NP	88.0	10.6
19.5	9	92.0	6.7
24.0	9	87.0	4.6

K. (Continued)

Field: Yuma Air Force Base
Facility: Taxiway 7
Assembly Load: 30,000 lb

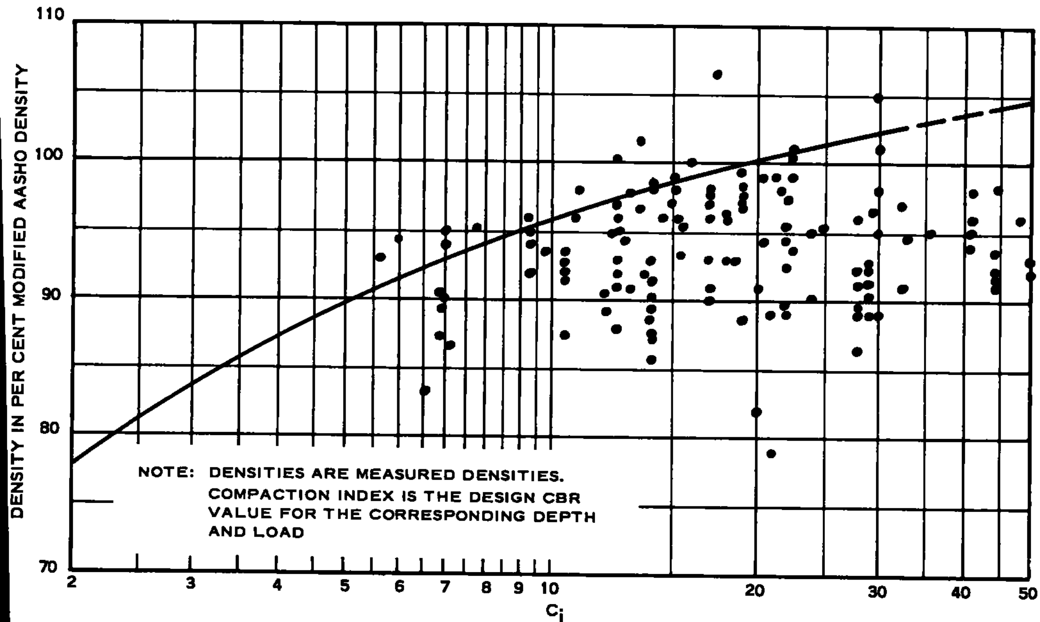
Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
8.0	NP	105.0	25.8
12.5	NP	103.0	14.5
24.0	NP	97.0	5.5
5.5	NP	104.0	38.7
12.5	NP	97.0	14.5
17.0	NP	94.0	9.6

Facility: N-S runway
Assembly Load: 30,000 lb
Assembly Type: Single, 100-psi tire pressure

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
6.5	NP	104.0	33.0
15.5	NP	99.0	11.0
24.0	NP	93.0	5.5
6.5	NP	103.0	33.0
14.5	NP	99.0	12.0
18.0	NP	101.0	8.8
5.0	NP	100.0	43.0
13.5	NP	96.0	13.3
24.0	NP	96.0	5.5
6.0	NP	103.0	35.7

K. (Continued)

Depth from Surface in.	Plasticity Index	Per Cent Mod AASHO Density	Compaction Index
15.5	NP	103.0	11.0
24.0	NP	89.0	5.5
6.5	NP	100.0	33.0
16.5	NP	95.0	10.0
24.0	NP	89.0	5.5
7.0	NP	98.0	30.0
16.5	NP	93.0	10.0
24.0	NP	89.0	5.5



'igure 4. Compaction requirements of cohesive (plastic) soils for flexible airfield pavements, Table 1 data.

ıd plasticity of the soil, and on the load, tire arrangement, tire pressure, and volume traffic. Table 3 summarizes the data from certain carefully controlled test sections; ese were considered of primary reliability. Table 4 summarizes data from airfields, ıich were considered of secondary reliability.

The data from Table 3 are plotted as diagrams of percent compaction versus com-ıction index in Figures 4 and 5. Since tolerable amounts of settlement from compac-ɔn have not been established, the points shown in Figures 4 and 5 cannot be separated to "acceptable" and "nonacceptable" categories with a dividing line drawn between em. The points in Figures 4 and 5 that plot toward the lower densities (for a given ɔmpaction index) represent cases where the amount of densification that occurred was ıall. This could easily be due to a low volume of traffic or a moisture content con-derably dry (or wet) of optimum. The points that plot toward the higher densities, wever, represent those cases where the volume of traffic was high and the moisture nditions were proper for compaction to occur. A limiting line, set high enough so ıt all points would fall below it, would be a completely safe limit; however, due to inaccuracies involved in density sampling and in determining the proper reference nsity (modified AASHO), it is felt that such a limiting line would be unduly conserva-e. Also, some of the points lying in high positions may be due to unusually high den-ies developed during construction, or to naturally high densities, rather than to traf-. The lines shown in Figures 4 and 5 are intended to exclude the majority of the ınts. The shape of the curves was influenced to some degree by the pattern of den-y-depth-load relations which was in use prior to the time this study was made.

In Figure 4, which treats cohesive soils, the material strength requirements and sultant normal design practices affect the values at high compaction indexes. Load-s applied to a test section or airfield that would plot in the high C_i range would pro-e failure unless the materials involved had unusually high strengths (CBR values). hesive materials at or near optimum moisture content do not normally have these ısually high strengths, but may have them at moisture contents well below optimum. ollows that the data which were obtained for cohesive materials at high values of C_i ıld not have been in the proper moisture condition to give maximum compaction.

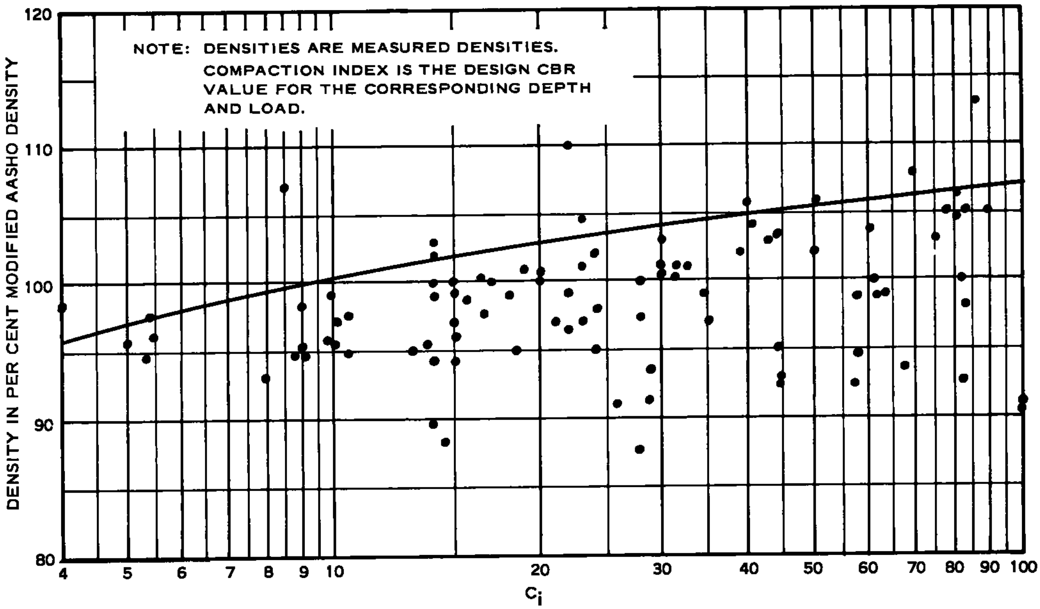


Figure 5. Compaction requirements of cohesionless (NP) soils for flexible airfield pavements, Table 1 data.

Therefore, data above a C_i of 50 have not been plotted, and some of the points immedi ately below a C_i of 50 must remain in question.

Figures 6 and 7 are plots of percent compaction versus compaction index for all th

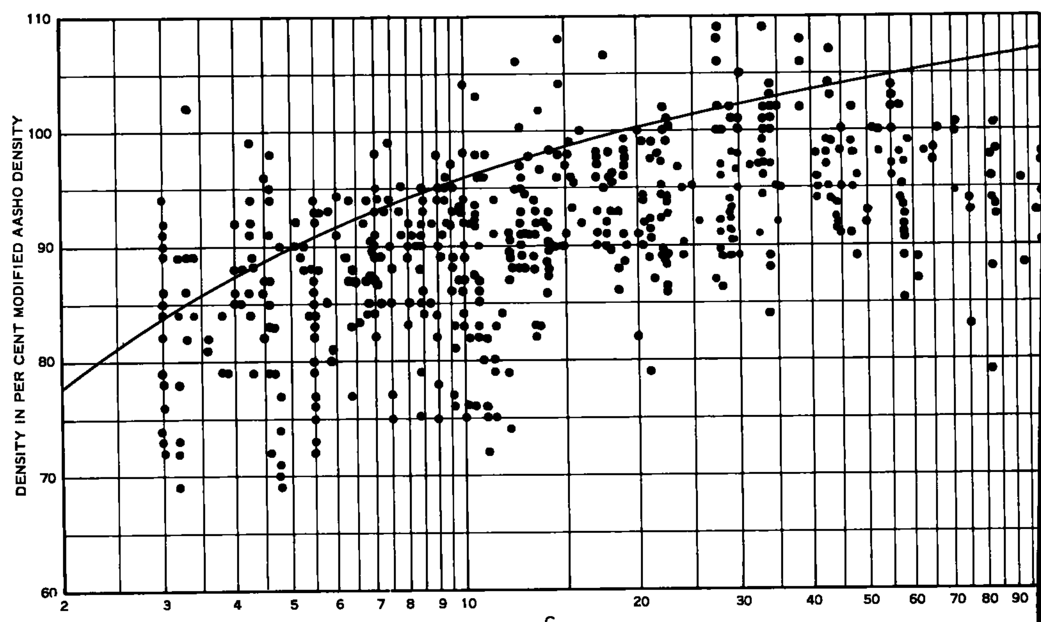


Figure 6. Compaction requirements of cohesive (plastic) soils for flexible airfie pavements, all data.

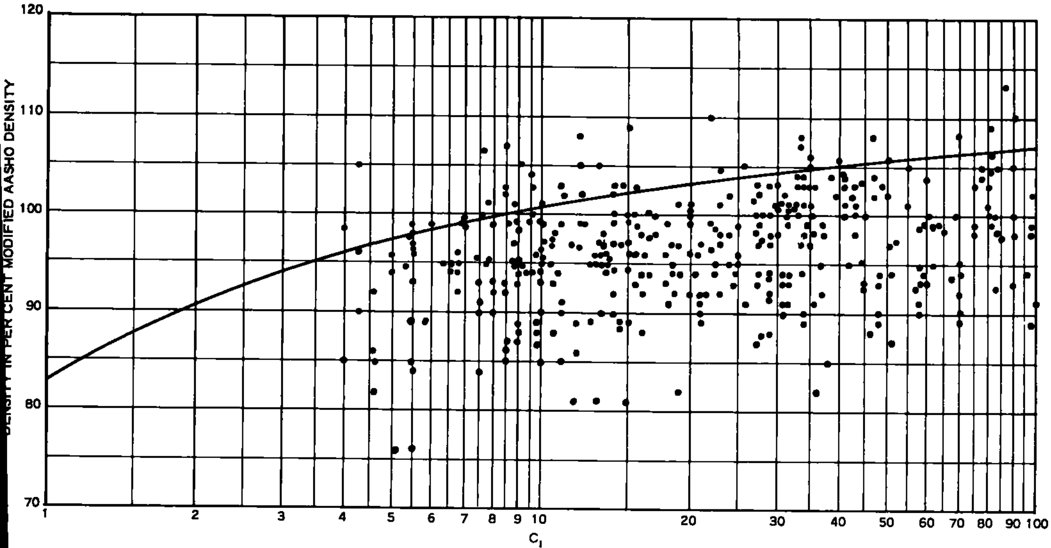


Figure 7. Compaction requirements of cohesionless (NP) soils for flexible airfield pavements, all data.

ata. The curves on these figures are the same as those shown in Figures 4 and 5. Vhile at first glance it may appear that Figures 6 and 7 are an unrelated scatter of oints, the plots have meaning if it is accepted that the required degree of compaction ecreases with decreasing compaction index. On this basis the uppermost points in ne right-hand portion in Figures 6 and 7 (the high C_i range) are considered to have reulted from compaction by aircraft traffic. On the other hand, densities indicated by ne uppermost points to the left were not necessarily the result of compaction by airraft traffic. For instance, 90 to 95 percent of modified AASHO maximum compaction s commonly required throughout fill sections, with 95 to 100 percent required in the op 6 in. of the subgrade. Also it is possible in some cases for cut sections to be at igher densities than those that will be produced by aircraft using the overlying pavenent. For these reasons, less importance should be attached to the high plotted points the left-hand portions of Figures 6 and 7. The absence of points indicating high denities in the very high C_i range in Figure 6 is due to the inability of cohesive materials exhibit these unusually high strengths at optimum moisture contents, as discussed reviously.

It was first thought that soil type as expressed by the plasticity index (PI) would be sufficiently critical parameter that it might be treated in a number of ranges, such

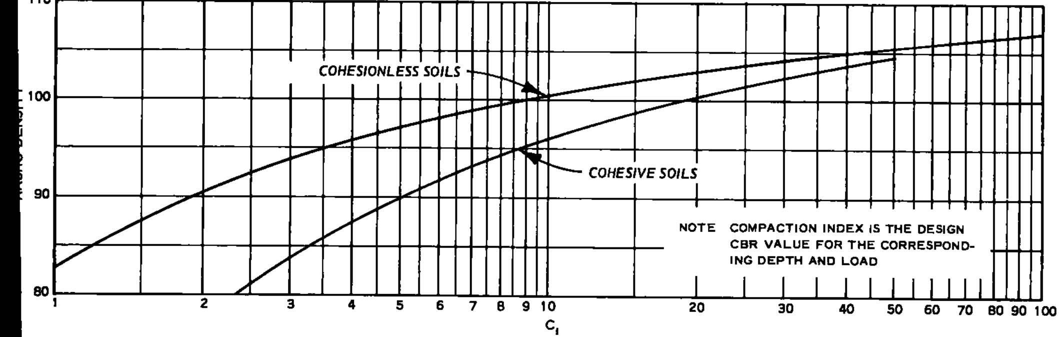


Figure 8. Compaction requirements for flexible airfield pavements.

TABLE 5

Material	Percentage Compaction
	Materials with Design CBR Values of 20 and Above
Base courses	Maximum that can be obtained, generally in excess of 100% of modified AASHO maximum and never less than 100%.
Subbases and subgrades	100% of modified AASHO maximum except where it is known that a higher density can be obtained practicably, in which case the higher density should be required.
	Materials with Design CBR Values Below 20
Select material and subgrades in fills	As shown below except that in no case will cohesionless fill be placed at less than 95% nor cohesive fill at less than 90%.
Subgrade in cuts	Subgrade in cuts must have natural densities equal to or greater than the values listed below. Where such is not the case, the subgrade must (a) be compacted from the surface to meet the tabulated densities, (b) be removed and replaced, in which case the requirements given above for fills apply, or (c) be covered with sufficient select material subbase and base so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.

Depth of Compaction for Select Materials and Subgrades

Type of Assembly	Gear Load, kip	Depth of Compaction in Feet for Per Cent Modified AASHO Compaction Shown: Cohesionless Materials				Cohesive Materials				
		100	95	90	85	100	95	90	85	80
Heavy Load Pavements										
Twin assembly, 37-in.	50	2	3-1/2	5-1/2	7	1	2	3	4	5
spacing, 267-sq-in.	100	3	5-1/2	7-1/2	10	2	3	4-1/2	5-1/2	7
contact area	150	4	6-1/2	9-1/2	12	2-1/2	4	5-1/2	7	8-1/2
Twin-twin assembly,	160	3-1/2	6	9	11-1/2	2	3	5	6-1/2	8
37-62-37-in. spacing,	240	4-1/2	8	11	15	2-1/2	4-1/2	6	8	10
267-sq-in. contact area	320	5-1/2	9	13	------	3	5-1/2	7-1/2	9-1/2	12
Light Load Pavements										
Single wheel, 100-sq-in.	10	1	1-1/2	2	2-1/2	1/2	1	1	1-1/2	2
contact area	20	1-1/2	2	3	3-1/2	1	1-1/2	2	2	2-1/2
	25	1-1/2	2-1/2	3-1/2	4	1	1-1/2	2	2-1/2	3
	30	1-1/2	2-1/2	3-1/2	4-1/2	1	1-1/2	2	2-1/2	3-1/2
Miscellaneous										
Single wheel, 100-psi	10	1	1-1/2	2	2-1/2	1/2	1	1	1-1/2	2
tire inflation	30	1-1/2	2-1/2	3-1/2	4-1/2	1	1-1/2	2	2-1/2	3
	50	2	3-1/2	4-1/2	6	1	2	2-1/2	3-1/2	4
	70	2-1/2	4	5-1/2	7	1-1/2	2-1/2	3	4	5

as nonplastic, 0-5 PI, 5-10 PI, 10-25 PI, etc. On analysis, however, it was found that distinctions could not be made between the various ranges of plasticity, and that only the separation into cohesive and co-hesionless (plasticity index zero or NP) was warranted. This finding was partly due to the small differences between ranges and partly to the data being insuf-ficient to establish such small differences.

The percent compaction versus com-paction index curves (shown for both soil ypes in Fig. 8) are the basis of the com-paction requirements shown in Table 5. These are the requirements contained in he current (Aug. 1958) Corps of Engi-eers' design manual for pavement areas ubject to normal traffic distribution. The ompaction indexes from Figure 8 were sed with the respective CBR design curve determine the depth to which the various egrees of compaction should be specified r subgrades with design CBR values less an 20. The depths are rounded off to e nearest half foot. As in previous is-ies of the manual, the minimum compac-on requirements for fills are specified 95 percent for cohesionless materials d 90 percent for other soils. These are relatively moderate compaction requirements. he values shown in Table 5 for 80 and 85 percent compaction are intended for use in e-luating the adequacy of the natural density in cut sections. Where the natural density less than the requirements, the soil must be compacted to the required density by lling from the surface of the cut (not effective unless the moisture content at the time of rolling is proper) or by removal and re-placement in lifts.

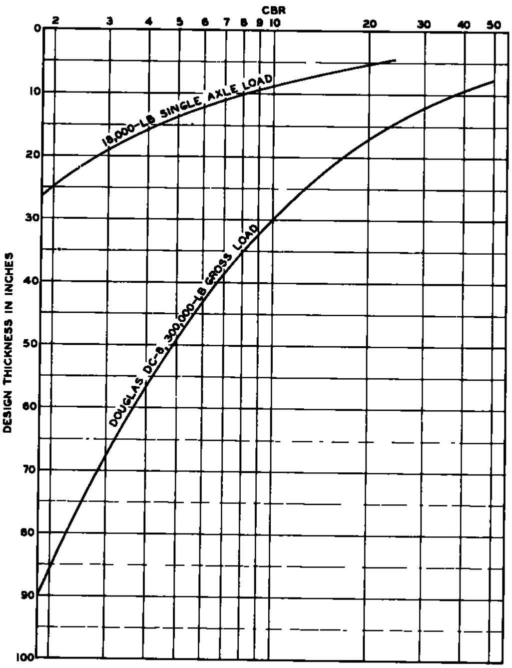
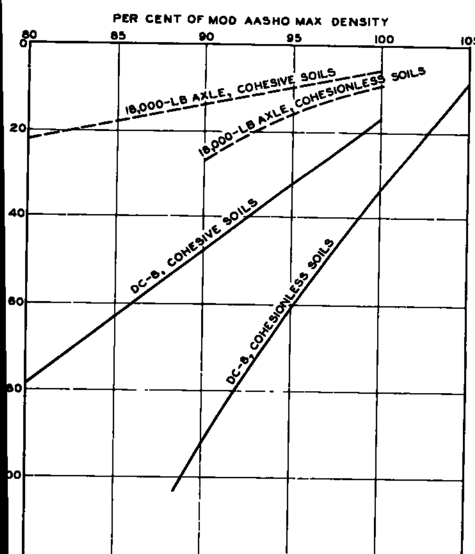


Figure 9. CBR design curves.



ure 10. Example of density require-ments.

As shown in Figure 8, indicated percent-age of compaction for a compaction index of 20 and above (design CBR of 20 and above) is in excess of 100 percent. Compaction requirements for materials with design CBR values in excess of 20 (base courses, sub-bases, and high-strength subgrades) are given in Table 5 in a narrative form, rather than as a table, to emphasize the necessity for high degrees of compaction for these materials.

The compaction requirements indicated by the compaction index apply only to the problem of densification by traffic. The problem of the consolidation produced in subgrades and foundations by high fills is a soil mechanics problem.

Application to Civil Airfields and Highways

Figure 8 can be used to establish com-paction requirements for civil airfields and for highways when CBR design curves are available. The procedures are illustrated by the following examples. Figure 9 shows CBR design curves for an 18,000-lb, single-axle load (from Fig. IV-2, very heavy traffic class, (3)), andfor a Douglas DC-8 plane at 300,000 lb (from Fig. 4, (1)). The compaction index in Table 6 was read from Figure 8, and the corresponding thickness from Figure 9. For example, the compaction index for 95 percent of modified AASHO maximum density from Figure 3 is 3.5 for cohesionless soils and 8.6 for other soils. The compaction index is converted directly to design CBR (compaction index of 3.5, design CBR of 3.5) and the thicknesses read from the proper curve in Figure 9. For example for the 18,000-lb axle load, the thicknesses indicated from Figure 9 are 17 in. for cohesionless soils and 10 in. for other soils.

TABLE 6

Compaction, %	Cohesionless Soils[1]			Cohesive Soils		
	Compaction Index	Thickness (in.) 18,000-lb Axle	Thickness (in.) DC-8	Compaction Index	Thickness (in.) 18,000-lb Axle	Thickness (in.) DC-8
105	42	-	9	-	-	-
100	9	10	32	19	6	17
95	3.5	17	61	8.6	10	33
90	1.8	27	92	5.0	14	49
85	-	-	-	3.2	18	63
80	-	-	-	2.4	22	79

[1]PI = 0.

Figure 10 is a plot of the percent compaction versus depth given in Table 6. Norm ly, the curves in Figure 10 would be used to establish a step-pattern of compaction re quirements. For example, for the 18,000-lb axle load, 95 percent of modified AASH maximum density would be required to a depth of 14 in. from the finished surface of t pavement, and 90 percent to a depth of 18 in., in cohesive soils. In cohesionless soi 100 percent of modified AASHO maximum density would be required to a depth of 15 i from the finished surface of the pavement, 95 percent to a depth of 27 in. The depth would probably be shifted an inch or two to coincide with a lift. Also, 95 percent wou probably be specified for all cohesionless fills, and 90 percent for other fills.

SUMMARY

The design CBR, termed the "Compaction Index," C_i, provides a means of combi ing into a single parameter the variables of load, tire arrangement, tire pressure, volume of traffic, and depth from the surface to the layer being studied. The relatio developed by the Corps of Engineers Flexible Pavement Laboratory, between compac tion index and the required percentage of modified AASHO maximum density are presented. These relations can be used to develop compaction requirements for civil ai field and highway loadings. Examples of the procedures are given.

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Discussion

EDWARD A. ABDUN-NUR, Consulting Engineer, Denver, Colorado—In developing design compaction requirements from the actual observations on compaction of the various layers in airfields subjected to actual and to accelerated traffic, the authors have given the profession a very realistic approach to design criteria—badly needed in this field. They are to be highly commended for such a fine piece of work.

Figures 4 and 5 are most interesting in that they form the basis of the relationship between compaction requirements and compaction index, from which the requirements at different depths for different wheel loads, arrangements and tire pressures are later derived. Figures 6 and 7 are still more interesting in that they contain a much larger population, even though part of it may not be as reliable as that in Figures 4 an 5. These figures represent, in essence, the basic data from which all the final relationships and conclusions in the paper are drawn.

The authors have very carefully and capably given various reasons and explanations for the scatter of the data exhibited in these figures. Additional reasons and explanations that have also been factors in this scatter, can no doubt be enumerated. Howeve irrespective of any reasons and explanations, this scatter must be accepted as a norm physical picture of any universe being studied. The very orderliness that the authors have implied must exist in the data, and which their explanations tried to justify, simp does not exist in nature or on any project.

With this in mind, the writer questions plotting the curves in these figures at what appears to be the 85 to 95 percentile of the universe. The effect of using such a high level for a basis of design is to inject a factor of safety that is not needed and that will unjustifiably increase the cost of facilities designed to such standards. If to this is added the fact that such levels obtained from 85 or 95 percentile points are further used as minima, then the additional factors of safety interjected by this mechanis lose their practical justification.

It seems to the writer that a realistic approach would be to fit a curve around the average or mean of the data. This automatically allows for the scatter which is boun to result in the compaction on any construction job. If the ultra-conservative curves shown in these figures and the resulting increased cost are justified by other considerations, then at least, the average requirement of compaction should be used instead o the minimum.

Control of compaction in a universe to a definite minimum is unrealistic, impractical, and nearly impossible of attainment on a construction project. The reasons for this have been developed by the writer for portland cement concrete in a paper delive ed at the 1961 Convention of the American Concrete Institute. They are just as applicable to soils, base courses, and bituminous concrete, except that the variations are of a different magnitude in each case. Control by maintaining an average compaction requirement that will assure a predetermined probability that no more than a predete mined percentage of the universe will fall below a given design figure is much more practical, represents the actual physical conditions on the job more realistically, an is obtainable. Such an approach has been used by the writer for several years, and h been recommended recently for compaction, as a result of the AASHO Road Test by W.N. Carey, Jr., J.F. Shook, and J.F. Reynolds in a paper presented at the 1960 Annual Meeting of the American Society for Testing Materials.

If such an average requirement is tied to the uniformity of a given contractor oper ation, a motivation can result that will improve the uniformity of the work far beyond that obtained by any degree of inspection.

W. H. CAMPEN, Manager, Omaha Testing Laboratories—Apparently the densities which are sufficient to produce required CBR values in subgrades, subbases and bases are not high enough to prevent further densification in the field by loaded tires. The authors therefore are proposing a method whereby the necessary degree of density can be specified for various depths of the layered systems under different wheel loads and tire pressures.

Based on the usual relationship between density and CBR the procedure recommended will result in higher values of CBR. Theoretically the thicknesses should therefore be reduced. Has this point been given consideration?

The writer notices also that the sandy or cohesionless subbases attain much higher densities, in respect to designed densities, than other types of subbases. In the writer's opinion the results are to be expected because it is well known now that the impact method used in the laboratory in making the moisture-density test gives low results on cohesionless materials. A comparison of the results obtained with the impact method with those obtained by the inundation-vibration method on ordinary sand may show the former to be only 92 of the latter.

C. R. FOSTER and R. G. AHLVIN, Closure—The authors agree that Mr. Abdun-Nur's proposal to use statistical quality control methods in the control of compaction is a good one. The Waterways Experiment Station has made limited use of such methods in research work involving repetitive density sampling. The Corps of Engineers, however, is not geared to use of such methods in connection with specification compliance determinations, and it will be some time before adequate service test trials and education of field personnel will permit their use.

In regard to the analysis in the paper being discussed, it is doubtful that the methods Mr. Abdun-Nur proposes should be applied. As Mr. Abdun-Nur points out, scatter is found to occur in the compaction on any construction job. The data being analyzed, however, are for a multitude of jobs and not just one. Essentially, each plotted point in the figures to which Mr. Abdun-Nur refers (4-7), represents a separate job and therefore a separate universe in regard to the type of control proposed. An attempt to apply the same methods to the universe of universes represented by the data involves random treatment of unknowns and uncontrolled variables of such magnitude that the variability is greater than the significant range in parameters. Also, such an attempt would result in an average which would apply to a collection of subsequent constructions such that half of these constructions would be satisfactory with a degree of conservatism ranging upward from none, whereas the other would be unsatisfactory, ranging from slightly to greatly unsatisfactory.

Although the authors do not believe the methods proposed by Mr. Abdun-Nur apply to their analysis, this in no way detracts from the merits of the methods, and one cannot fail to recognize their advantages in regard to construction control.

Mr. Campen's question hews directly to the practical aspects of the interrelation of strength (CBR) and density, and reflects his intimate knowledge of the subject. A design CBR value must be determined for each material used in a pavement structure, and design values necessarily depend on the density to be attained. It is, or has been, common practice to select design values from laboratory CBR test results based on a given percentage of a standard density—frequently 90 or 95 percent of modified AASHO maximum density. Mr. Campen points out that where a higher density is required, a higher design CBR value may be selected.

Corps of Engineers' procedures specify a determination and plotting of CBR test results for a range of moisture contents, densities, and compactive efforts from which design CBR values are selected. Plots of data of this type permit selection of CBR design values for any pertinent values of moisture content and density.

The authors are glad to have Mr. Campen's comment on the agreement of his experience with theirs in regard to the ready attainment of higher densities in cohesionless materials.

An Analysis of Hybla Valley Rigid Plate Bearing Data

G. RAGNAR INGIMARSSON, Research Assistant, Soil Mechanics Laboratory, University of Michigan, Ann Arbor

This paper presents an analysis of some 89 rigid plate bearing tests, on 26 different flexible pavement sections at the experimental test track at Hybla Valley, Va. The test data are those reported by Benkelman and Williams (1, Tables 4 and 7). The linear equation developed by W.S. Housel (2) is used in the analysis. Statistical results indicating the accuracy with which this linear equation reproduces the results of bearing capacity tests on different sizes of plates are presented. The analysis is carried to the point of determining the stress reactions developed by the flexible surfaces and the supporting subgrade; these results are presented graphically. Bearing capacity and resistance factors for different thicknesses of base and surface are compared. Use of a high-speed digital computer in this analysis is described. Also presented are methods of programming and a cost analysis.

● HRB Special Report 46 (1) contains data from rigid plate bearing tests carried out at the experimental test track at Hybla Valley, Va. Four different test procedures were employed; namely, the incremental, the incremental repetitional, the accelerated, and the repetitional.

The following analysis has been limited to the accelerated tests only. The data fro[m] this test procedure were chosen because they provide a larger variety of pavement sec[-]tions, subjected to a wider range of loadings, than do the other test data. Furthermo[re,] this test series is the only one in which a uniform rate of loading was maintained thro[ugh]out the series, permitting a valid comparison between load and settlement of different plate sizes and pavement thickness.

The symbols and abbreviations used in this paper are as follows:

- A = area of plates in square inches;
- B = thickness of stabilized aggregate base in inches;
- D = diameter of plates in inches;
- K_1 = settlement coefficient ($\frac{\Delta}{n}$);
- K_2 = stress reaction coefficient ($\frac{m}{n}$);
- m = perimeter shear in pounds per inch (pi);
- n = developed pressure in pounds per square inch (psi);
- P = perimeter in inches;
- p = unit load or bearing capacity in pounds per square inch (psi);
- t = total pavement thickness in inches;
- W = total load in pounds;
- Δ = deflection or settlement in inches;
- A.C. = thickness of asphaltic concrete in inches; and
- Rem. = removed.

The accelerated test procedure consists of two parts, designated as the increment[al] portion and the accelerated portion. The first part provides for application and rele[ase] of three individual loads of increasing magnitude, the period of application or releas[e] being maintained until the rate of movement slows down to 0.001 in. in 15 sec. Foll[owing]

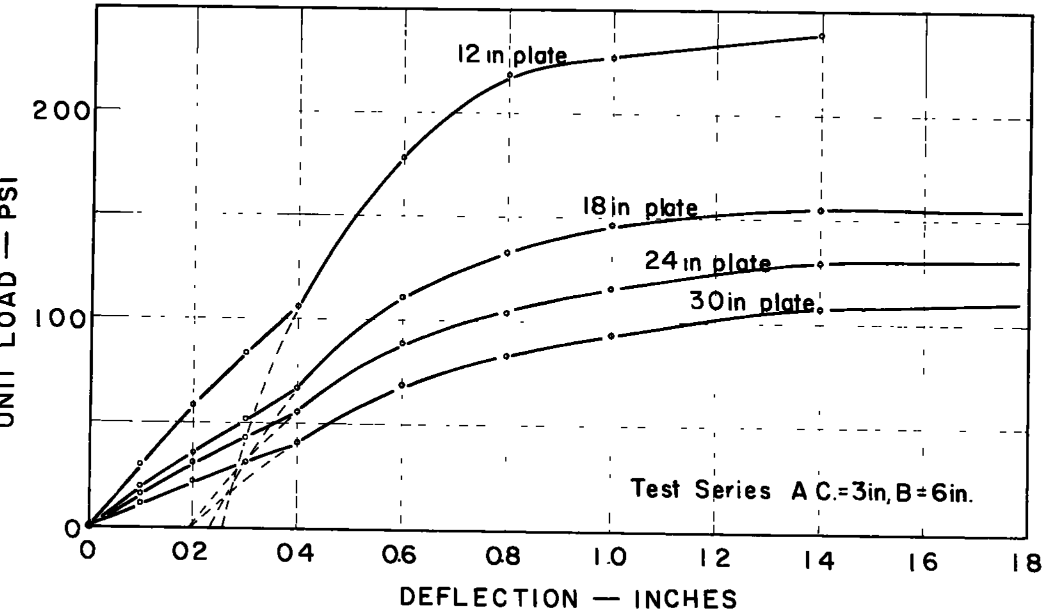

Figure 1. Load-deflection graph.

ng the release of the third load, the accelerated portion is carried out, providing for rate of vertical movement of the surface under a load applied at a settlement rate of .5 in. per min.

Figure 1 shows a typical load-deflection graph from the accelerated tests. As ex-ected, there is a definite discontinuity in the graph at 0.4-in. deflection, due to the hange in rate of loading.

THE LINEAR EQUATION

In Housel's perimeter-shear theory (3), the bearing capacity or intensity of load is xpressed by the following straight line equation for a given amount of deflection:

$$p = m \frac{P}{A} + n$$

which

- p = unit load or bearing capacity;
- m = perimeter shear, load per unit length;
- n = developed pressure, load per unit area;
- P = perimeter; and
- A = area.

Figure 2 shows how a soil mass devel-s resistance to applied load in terms of rimeter shear, m, and developed pres-re, $n_1 + n_2$. It will be noted that all the d applied to the surface of the soil o-inates within the plate area. Below the rface some of the load is then distribu- laterally as perimeter shear and the mainder transmitted directly down the ntral column as developed pressure.

Previous investigations of plate loading

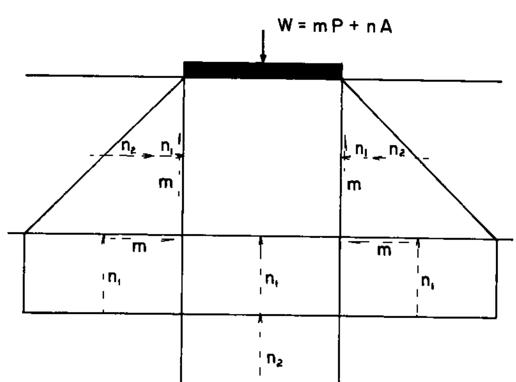

Figure 2. Stress reactions in cohesive soil.

tests have shown that the magnitude and sequence in which these stress reactions are developed varies widely, depending on the relative rigidity of the bearing plate and supporting elements of the soil mass. In the normal case the perimeter shear and developed pressure are mobilized simultaneously, with both having positive magnitudes throughout the entire range of load and settlement. In relatively compressible materials the perimeter shear reaches limiting values first and developed pressure, indicated by concentration of pressure in the central column, follows as the final limit of supporting capacity.

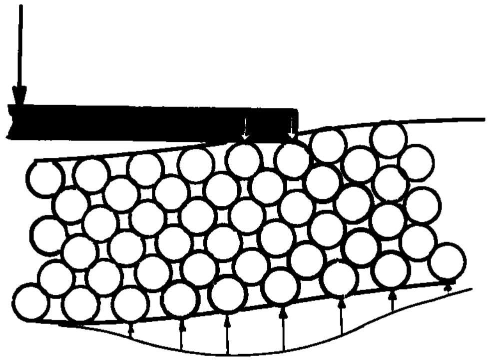

Figure 3. Pressure transmission through pavement.

In layered systems, such as a flexible pavement, it has been found that the sequence in which the two basic stress reactions are developed is the same, but that the rates at which they are mobilized are controlled by the relative rigidity of the bearing plates and supporting elements of the pavement structure and subgrade (4). As the load is applied, an elastic depression forms under the bearing area; rigid plates tend to bridge this depression (Fig. 3) where the transmission of pressure concentration at the edge of the plate through granular paving mixtures has been visualized in terms of arching action. Similar pressure distribution takes place through cohesive mixtures where shearing resistance is the basic reaction.

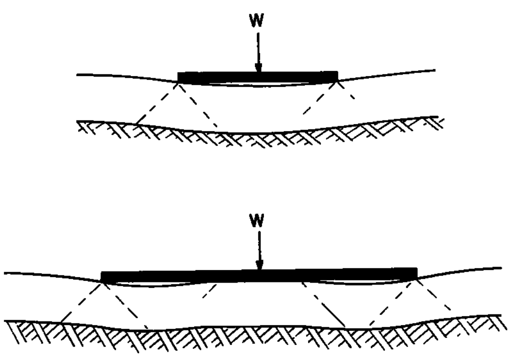

Figure 4. Deflection of pavement under various sizes of plates.

Pressure transmission through a flexible pavement structure is also influenced by the size and rigidity of the bearing plate (Fig. 4). In larger plates where pressure transmission from the perimete is limited in magnitude or angle of pressure transmission from affecting the central zone, direct transmission of pressur down the central column becomes a facto These variations in pressure transmissio must be included in the dimensional effec in plate loading tests and in their analysis in terms of the linear equation for bearin capacity.

The first question is whether or not it is possible to express the bearing capacity of flexible pavements by this linear equation. The second question is whether or not the stress reactions in this type of an alysis will reveal the significant structural behavior of flexible pavements, in spite o the variations which may occur in the sequence and magnitude of these reactions.

ANALYSIS OF DATA

As a first step in the analysis of the test data, it was decided to investigate how well the linear equation represented the relationship between the bearing pressures on the various plate sizes at a constant settlement.

In reviewing the typical load-deflection graph (Fig. 1), involving two different rate of loading, it was obvious that it would be necessary to treat the two portions of each load-deflection curve separately. To do this, it was necessary to estimate the no-lo deflection value for the two portions of each curve. Inasmuch as the primary objecti of loading tests is to determine the ultimate supporting capacity of the flexible pavements, further analysis was concentrated on the higher ranges of load and the initial

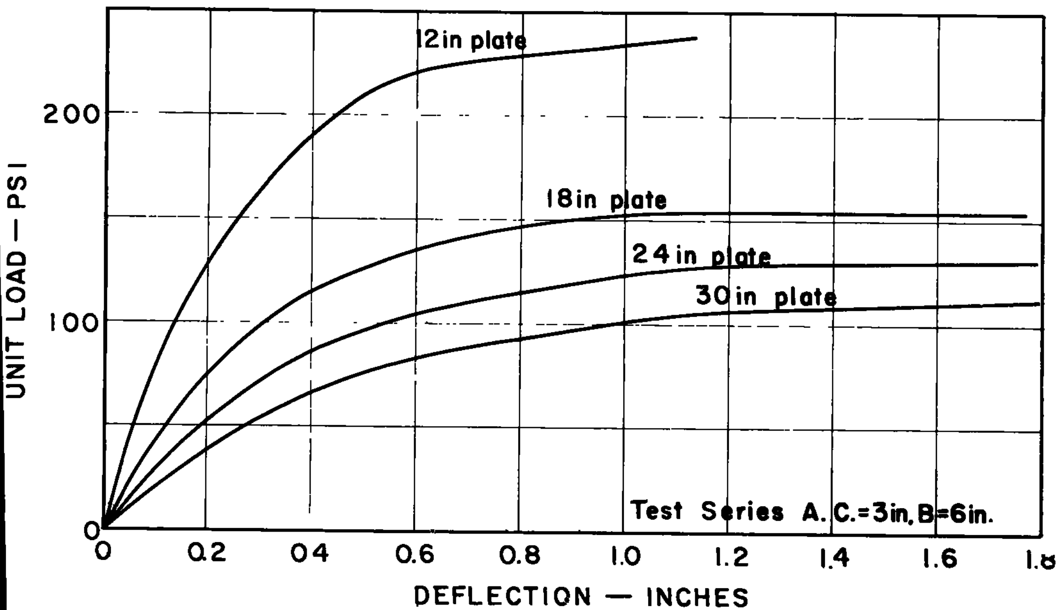

Figure 5. Adjusted load-deflection graph.

TABLE 1

COMBINATIONS OF PLATE SIZES TO WHICH THE LINEAR EQUATION WAS APPLIED

Pavement Sections	Plate Diameters (in.)		
	12-18-24-30	12-18-24	18-24-30
3-in. A.C. - 0-in. Base	X	X	X
3-in. A.C. - 6-in. Base	X	X	X
3-in. A.C. - 12-in. Base	X	X	X
3-in. A.C. - 18-in. Base	X	X	X
3-in. A.C. - 24-in. Base	X	X	X
6-in. A.C. - 0-in. Base	X	X	X
2-in. A.C. - 0-in. Base	X	X	X
6-in. A.C. - 6-in. Base		X	
6-in. A.C. - 12-in. Base		X	
6-in. A.C. - 18-in. Base			X
6-in. A.C. - 24-in. Base			X
9-in. A.C. - 6-in. Base		X	
9-in. A.C. - 12-in. Base		X	
9-in. A.C. - 18-in. Base			X
3-in. A.C. Rem. - 6-in. Base	X	X	X
3-in. A.C. Rem. - 12-in. Base	X	X	X
3-in. A.C. Rem. - 18-in. Base	X	X	X
3-in. A.C. Rem. - 24-in. Base	X	X	X
6-in. A.C. Rem. - 6-in. Base		X	
6-in. A.C. Rem. - 12-in. Base		X	
6-in. A.C. Rem. - 18-in. Base			X
6-in. A.C. Rem. - 24-in. Base			X
9-in. A.C. Rem. - 6-in. Base		X	
9-in. A.C. Rem. - 12-in. Base		X	
9-in. A.C. Rem. - 18-in. Base			X
9-in. A.C. Rem. - 24-in. Base			X

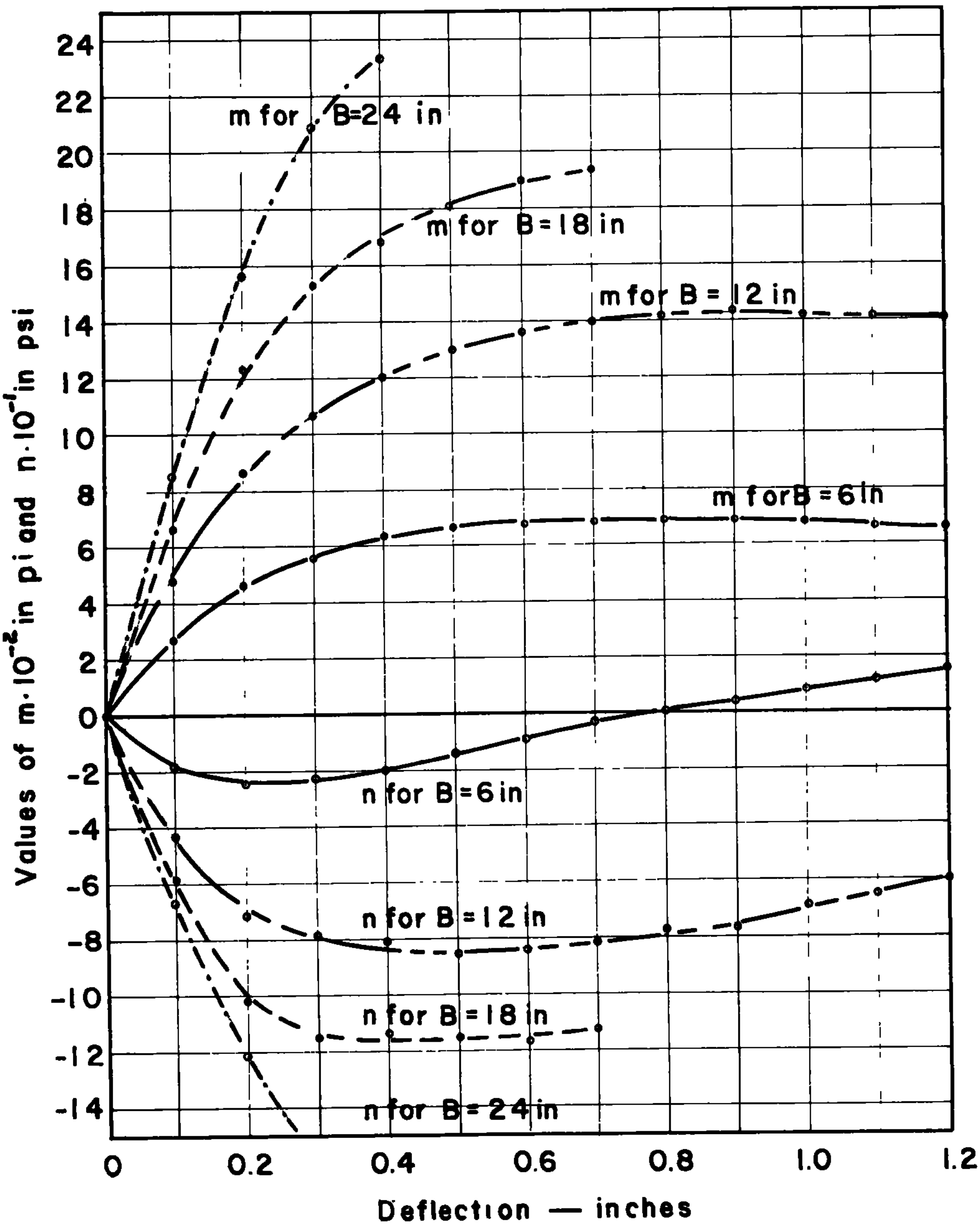

Figure 6. Values of m and n for 3-in. A.C. surface.

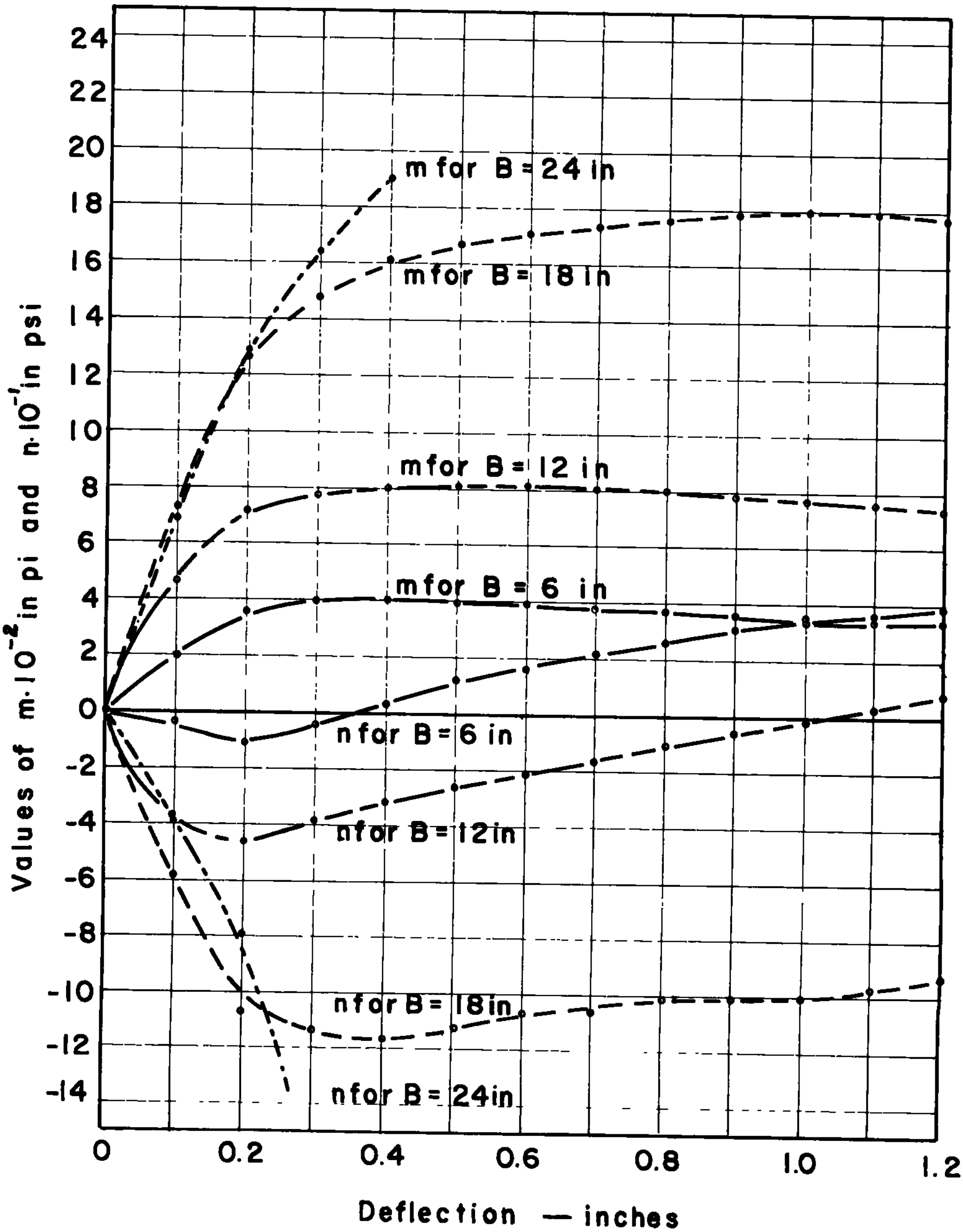

Figure 7. Values of m and n for 3-in. A.C. removed.

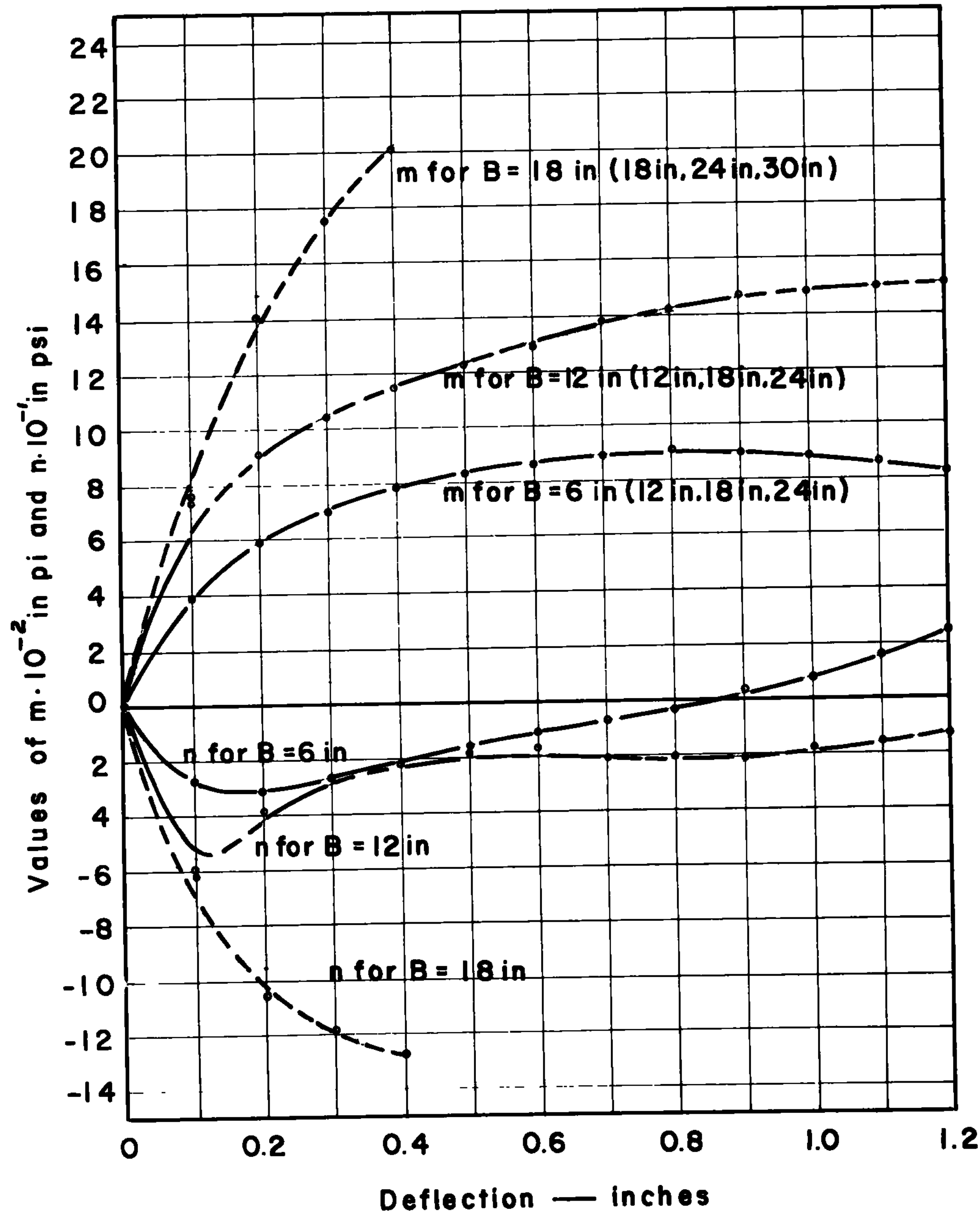

Figure 8. Values of m and n for 6-in. A.C. surface.

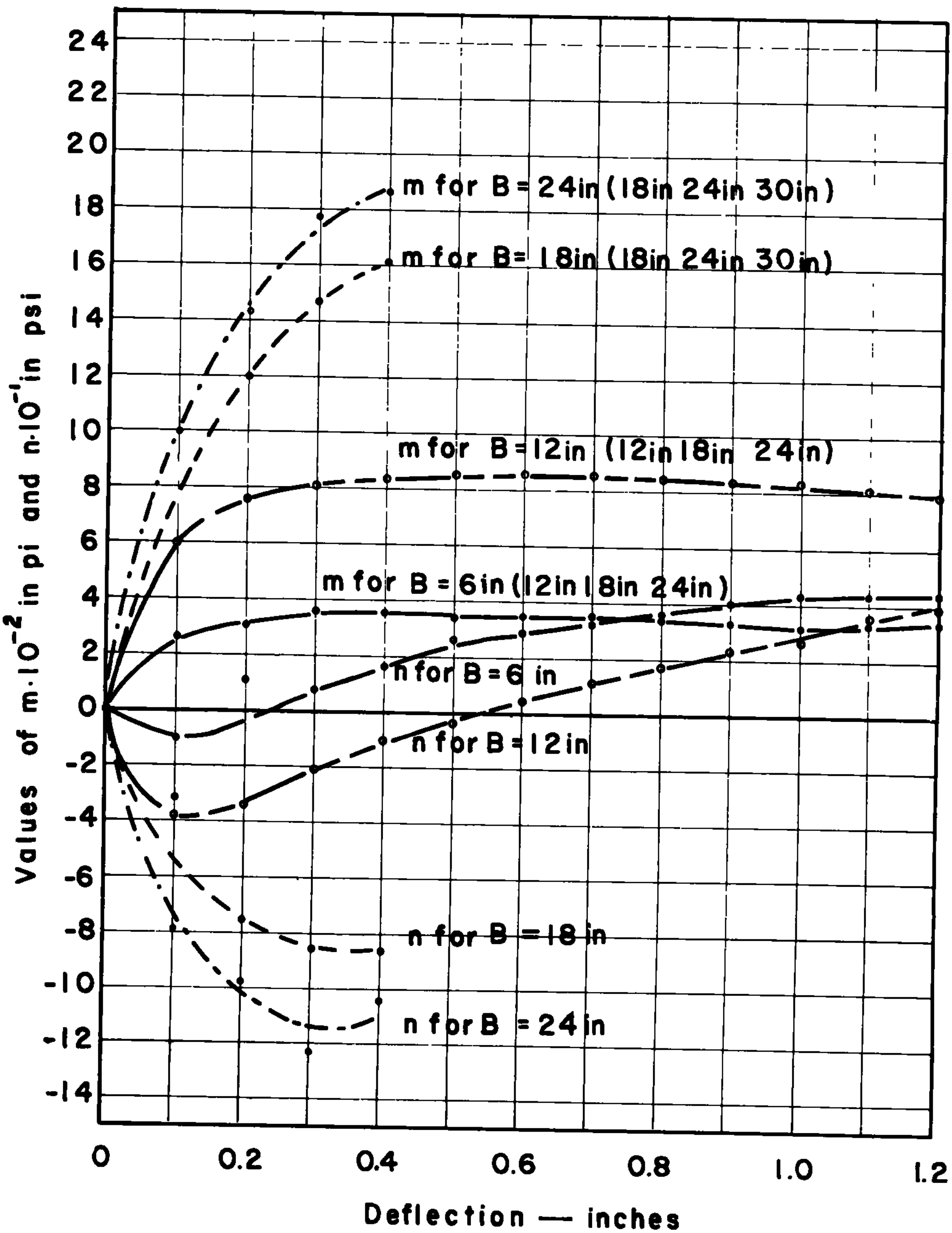

Figure 9. Values of m and n for 6-in. A.C. removed.

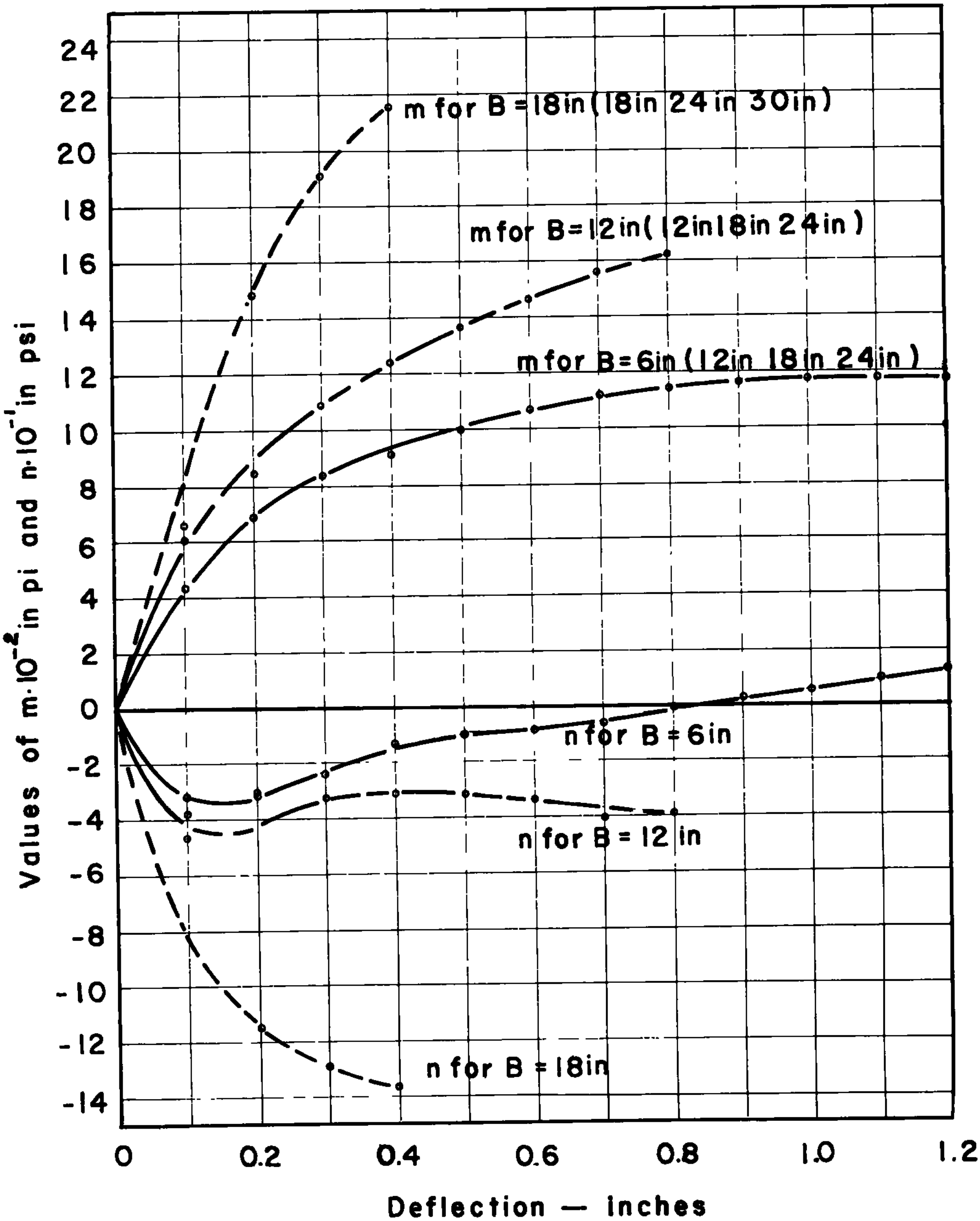

Figure 10. Values of m and n for 9-in. A.C. surface.

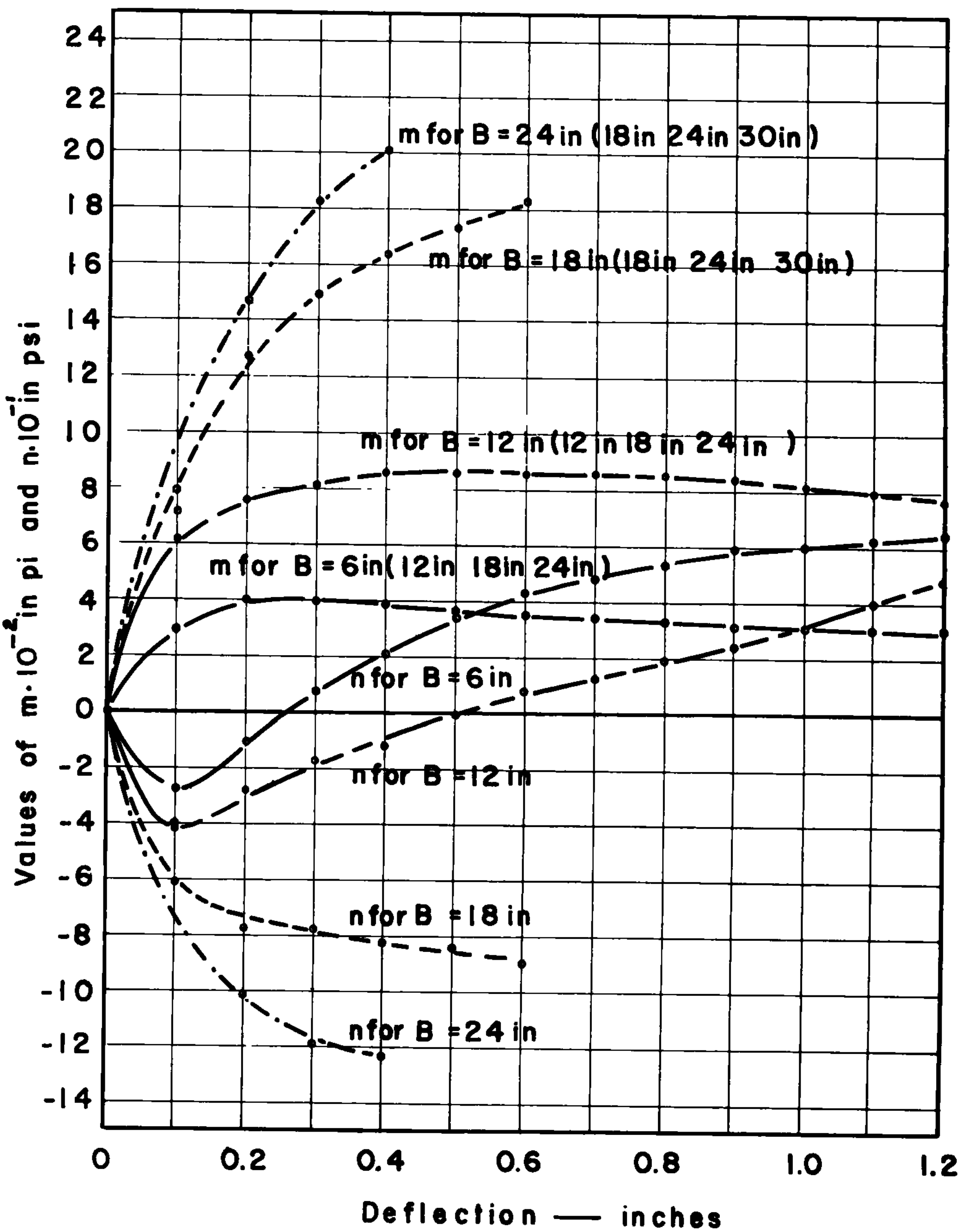

Figure 11. Values of m and n for 9-in. A.C. removed.

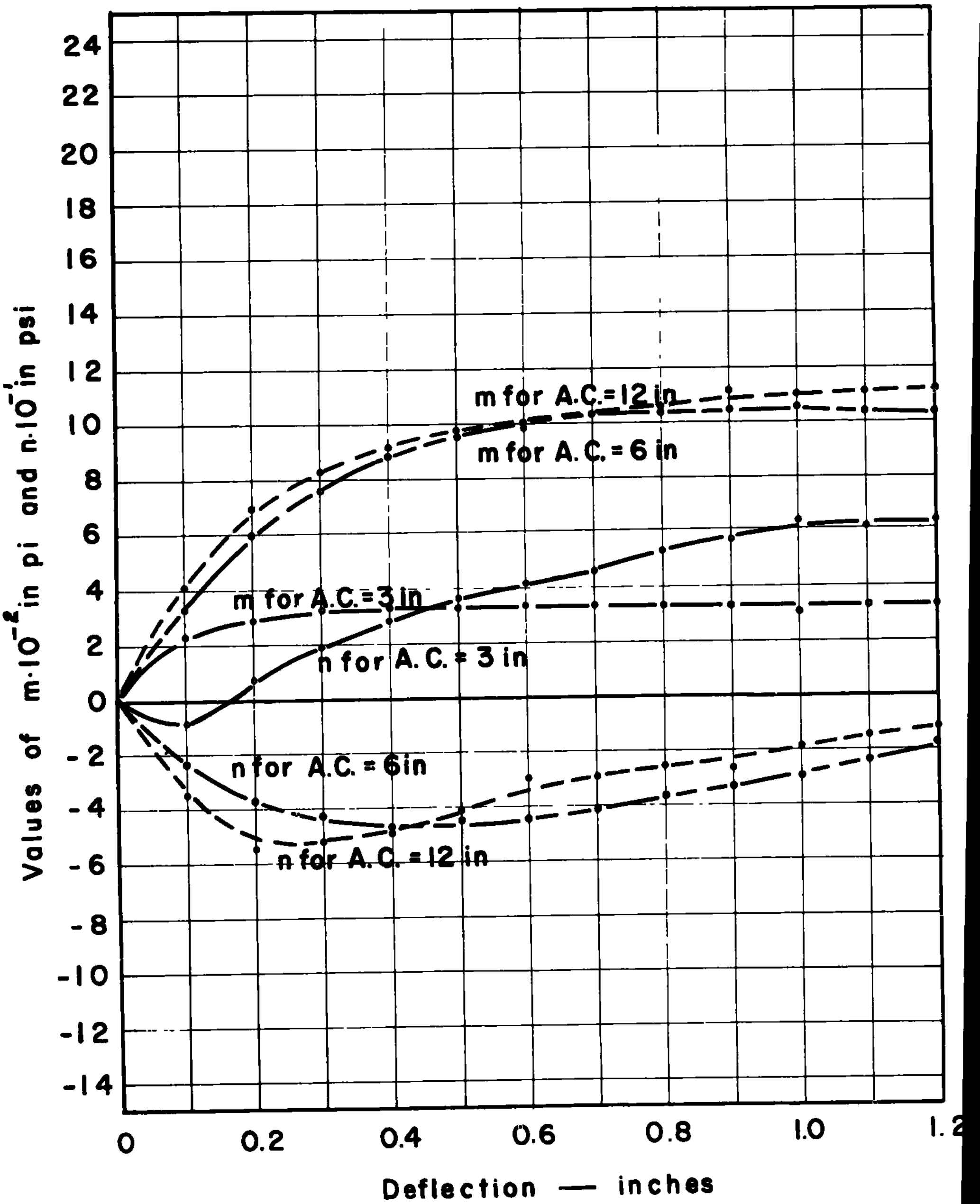

Figure 12. Values of m and n for variable A.C. surface and B = 0 in.

repetitive loading cycle of the accelerated test procedure was considered as a seating process for the accelerated loading which followed.

The no-load deflections for the second portion of the curves could be decided on, either by extending the upper part of the curves graphically down to the abscissa or by considering the permanent settlement of the pavement after release of the last repetitive load as the no-load deflection.

Values obtained by the second method were used throughout the analysis; but, in most cases, both methods gave practically identical values.

In Figure 5 the load-deflection diagrams for the accelerated loading from Figure 1 have been reproduced with a common origin, hereafter referred to as zero deflection.

When all the test data given in Table 4 and Table 7 of HRB Special Report 46 had been treated as explained previously, the linear equation was tested for its capability to express the bearing capacity for various plate sizes at constant deflection. The method of least squares was used to determine the constants, m and n, in the linear equation.

It was realized in the beginning of the analysis that it would be advantageous to use a high-speed computer to carry out the numerical work. For this purpose, the author wrote a program for the IBM 704 high-speed digital computer. Details of the program are explained in the Appendix.

The linear equation was applied to three or four plates according to the available data for each pavement section. Table 1 gives all the pavement sections and plate sizes analyzed together as indicated.

The values for the stress reactions, m and n, obtained from the foregoing analyses are plotted in Figures 6 through 12 for base course thicknesses shown on each curve. In some cases, the values of m and n were obtained from three plates only, as indicated on the graphs. Values of m and n for the same thickness of asphaltic concrete surface but with varying base thickness are grouped together, except in Figure 12 where results are shown from three pavement sections with varying thickness of asphaltic concrete laid on the subgrade with no base course.

When the values of m and n in all test series had been obtained, the bearing capacity expressed by the linear equation was computed and compared to the measured values. Deviations of the computed bearing capacity were expressed as percentages of the measured values, and are presented in Figure 13 with percent of deviation as the abscissa and the percentage of almost 2,000 cases as the ordinate.

DISCUSSION OF TEST RESULTS

As summarized in Figure 13, the agreement between the test results and bearing capacity at constant settlement computed by the linear equation is remarkably good. All combinations of plate size and pavement thickness are represented in the statistical analysis; and, without exception, fall within the narrow range of experimental error shown. Ninety-two percent of all values fall within ± 5 percent, and 99.6 percent fall within the limits of ± 10 percent. Considering normal variations in construction practice, such results also demonstrate the excellent quality control exercised in the mixing and placement of paving materials and in subgrade preparation.

The data speak for themselves in answer to the first question, the validity of the linear equation as a measure of the variation in bearing capacity with the size of loaded areas in the case of flexible pavements. The second question, whether or not the stress reactions in this equation can be broken down into factors which reflect significant variations in the structural behavior of flexible pavements, is much more involved. A review of the data in Figures 6 through 12 brings out several strong trends which are consistent throughout the entire test series. Nevertheless, the complete interpretation of these stress reactions has proved to be peculiarly complex. In all cases, there is a large increase in the perimeter shear, m, as the pavement thickness is increased. This is perhaps quite obvious and could be anticipated. However, the magnitude of this increase is surprising and leads to other variations more difficult to explain.

TYPICAL LINEAR EQUATIONS

Figure 14 shows a set of linear equations for a typical test series for deflections of 0.2, 0.78, and 1.2 in. The plotted points show the accuracy with which the linear equation for bearing capacity reproduces the test results, illustrative of the data in Figure 13 for the entire series of tests. At the lowest deflection, 0.2 in., the bearing capacity is negative for the larger sizes of plates. This indicates that the larger plates will not develop positive supporting capacity until the pavement deflection or settlement exceeds that amount. Intercepts on the vertical axis give the values of developed pressure, n, at the indicated settlements. Negative values of n in the lower settlement

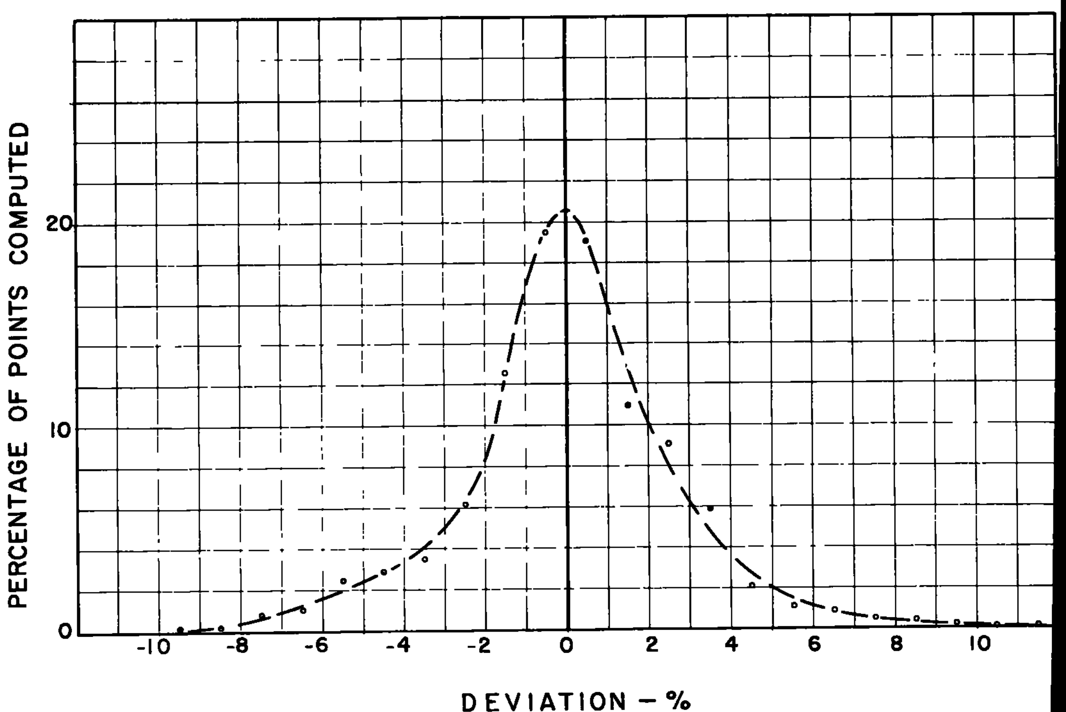

Figure 13. Percentage deviation of computed and observed bearing capacity.

range show that in this range the pressure is not being transmitted directly to the sub grade over the entire bearing plate. Such negative values of n are associated with hi values of perimeter shear, m, represented by the steeper slope of the straight lines Figure 14.

This variation in the stress reactions, m and n, shows that applied loads in the lo er range of settlement are being carried by pressure concentration at the edge of the bearing plates. This pressure concentration is then transmitted through the flexible pavement to the subgrade, where a substantial part of the perimeter shear will have been converted into developed pressure over the central column. Such results are n new, having been reported previously with partial explanations offered (4). Factors believed to produce these results have been shown in Figures 3 and 4 and discussed i a preliminary way. However, it is the quantitative evaluation of these reactions that presents the difficult problem that has yet to be resolved.

The relation between load, settlement, and size of bearing area has been formula in more general terms involving two soil resistance coefficients, K_1 and K_2 (3). The settlement coefficient, K_1, has been defined as the ratio of settlement, Δ, divided b developed pressure, n ($K_1 = \Delta / n$). This coefficient is analogous to the conventional

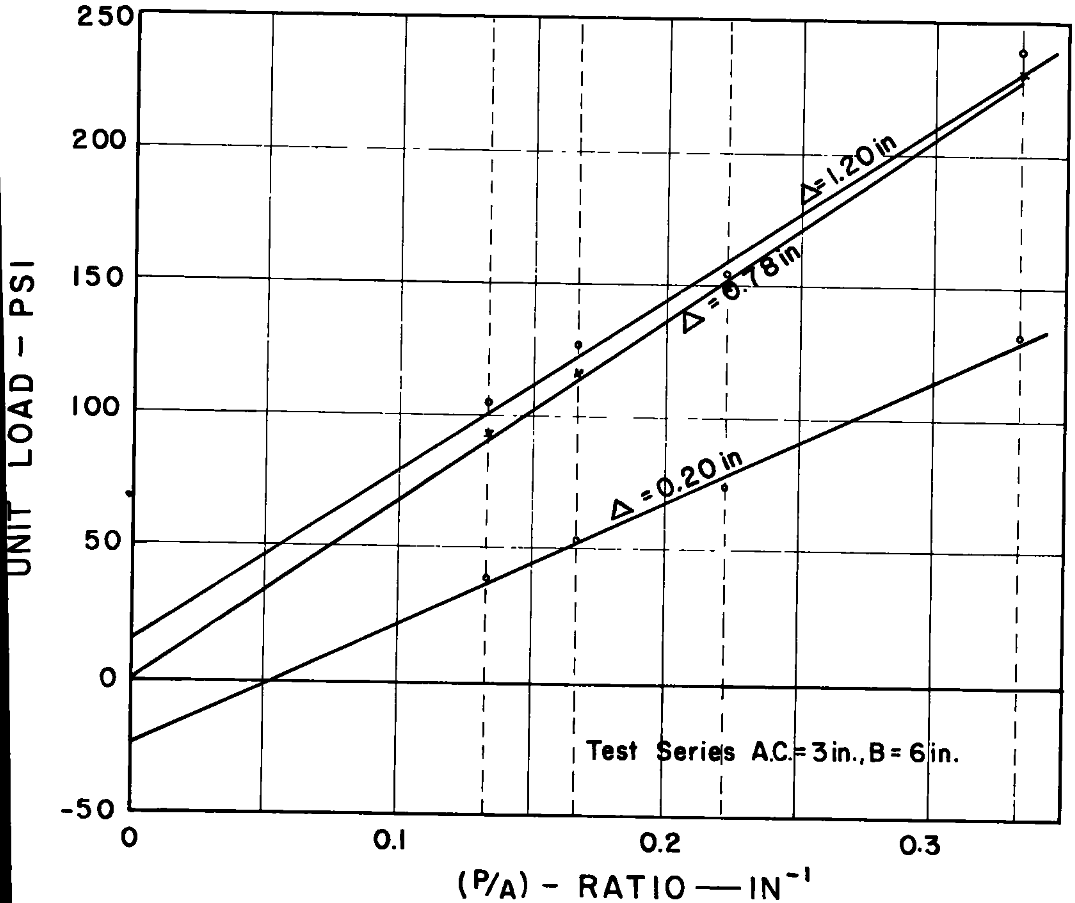

Figure 14. Typical linear equations.

oefficient of compressibility. The stress reaction coefficient, K_2, has been defined s the ratio of perimeter shear, m, divided by developed pressure, n ($K_2 = m/n$). K_2 ives the relative magnitude of these two types of resistance at any specified settle-ent.

Maximum and minimum values of the soil resistance coefficients, K_1 and K_2, have een identified as measures of the bearing capacity limit of supporting masses in terms static resistance. As shown in Figure 15, such maximum and minimum values oc-r in tests on flexible surfaces when the developed pressure, n, is equal to zero. hen encountered in previous tests, another method of identifying the static resistance mit was available for confirmation. This confirmation was provided by extrapolating tes of settlement for various loads to obtain the yield value or load at which progres-ve settlement was zero. Incremental loading at constant time intervals was not used the Hybla Valley tests, hence this demonstrated procedure is not available.

In passing, it may be noted that the ultimate capacity of these surfaces is such that e total loads employed in the investigation provided only a limited range of pavement flection which was not sufficient to reach limiting values of the variables involved. ttlement for the 24-in. pavement thickness seldom exceeded 0.4 in., and most of e tests for the 18-in. pavement are also limited in the settlement range. Several sts on the 24-in. base thickness have been omitted as there were only one or two ints on the load-settlement diagrams, not enough to justify plotting.

The present tests produce the largest volume of comprehensive data confirming se more complex variations that has yet been available for study; the factual na-

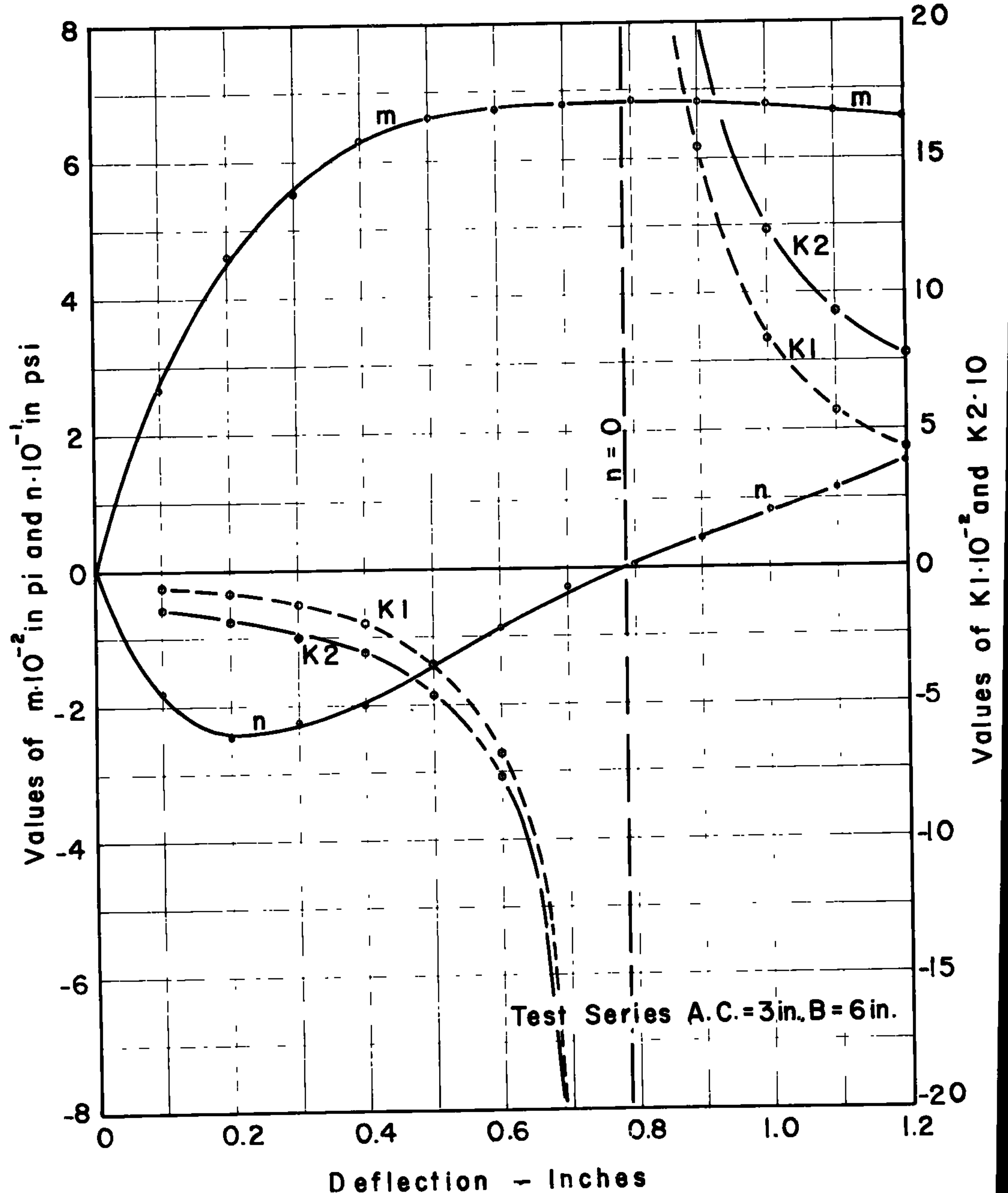

Figure 15. Soil resistance coefficients.

ture of these data cannot be passed over lightly. The extended range over which nega- tive values of developed pressure occur is surprising and this, too, is a consistent re- sult in all test series. In only a limited number of the tests has the loading been suf- ficient to produce a zero value of n, previously identified as the limit of static resis- tance in the pavement structure. However, there are a sufficient number of tests car- ried to and beyond this critical range to provide a fairly adequate basis for further an- alysis.

It is hoped that such further study may throw some light on the source and charact of these secondary effects. One possible approach that might be helpful is the non-dimensional analysis presented by Kondner and Krizek (5). It is hoped that these investigators may follow up this suggestion and see what their analysis might contribute to a solution of the problem. Housel has been following the author's work on the analysis of the loading tests from Hybla Valley, and presents a written discussion hereinafter. Perhaps others may come forward with other methods of analyzing these tests. The volume of data made available and the care with which it has been gathered have not been achieved in any previous investigation. Furthermore, the consistent variation in the stress reactions developed certainly justifies much more study on such an important problem in the design of a flexible pavement, the structural action of which is still quantitatively indeterminate in terms of the mechanics involved.

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Appendix

USE OF HIGH-SPEED DIGITAL COMPUTERS

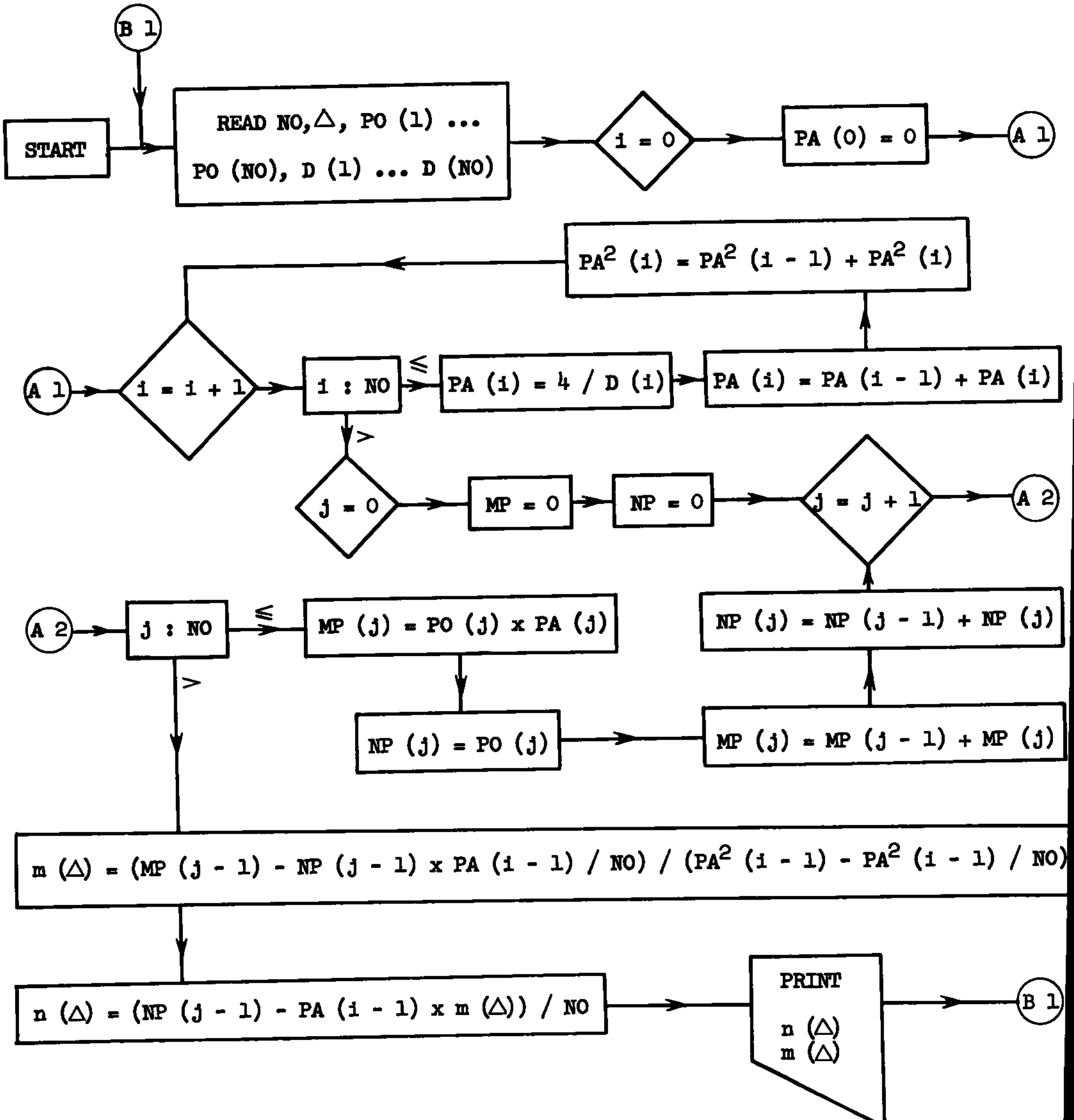
It may be assumed that in the near future there will be a very substantial increase n the use of high-speed digital computers in practically every field of engineering. roblems involving time-consuming computations, which are repeated over and over gain, are particularly adaptable to the use of high-speed computers.

Because the analysis of plate load bearing tests is at least partly this type of probem, the author took advantage of this opportunity and wrote a program which would ermit the use of a digital computer in carrying out the bulk of the numerical work.

A simplified flow-diagram which could be used for the evaluation of the stress retions, m and n from a set of data is shown in Figure 16. The flow-diagram is a raphical representation of the sequence of operations required to solve the problem question. It is absolutely independent of the computer or computer language used, t serves as a guide when one wishes to write a detailed program for a computer. or those not acquainted with this representation, it may be helpful if the two symbols " and "=" are defined. The symbol ":" means "Compare the variable on the left to e one on the right and choose between greater than (>) or less than (<), as indited." The symbol "=" means "Make the value of the variable on the left equal to e current values of the terms on the right."

The IBM 704 computer which was available is a large-scale computer which emys a special user's language called MAD, the Michigan Algorithm Decoder. The ogram was written in such a manner that it would be required only to feed the comter with the very minimum of information necessary to carry out the computations; d, when completed, the results would be printed or plotted in the most convenient rm.

Figure 17 shows a part of a data-deck which was used in this program. The first rd contains a title to be printed with the results. This may be any phrase the user ooses, containing no more than 80 letters and blanks. The second card contains me information pertaining to the computations themselves. The word "ROUND" inates that the plates used are round, and could be replaced by "SQUARE" or "RECNGLE." "DIMENSIONS" tells that the size of each plate is given in terms of dieter or sides, rather than "AREA." The next three numbers indicate the number



△ = Deflection

PO = Unit Load Observed

NO = Number of Plates Used

D = Diameter of Plates

PA = Perimeter-Area Ratio

Figure 16. Flow diagram for solution of stress reactions m and n.

of plates used, the number of deflection points to be computed, and the thickness of flexible pavement, respectively. "NO" means that it is not desired to call in the plot routine to produce a graphical representation of the results. The last two words indicate the units used. The third card gives the plate sizes, and the observed data are listed on the following cards. The data are listed as the value of deflection followed by the unit pressure for each plate; for example, at 0.1-in. deflection, 63 psi, 42 psi, and 31 psi, for the 12-, 18-, and 24-in. plates, respectively. If the next test

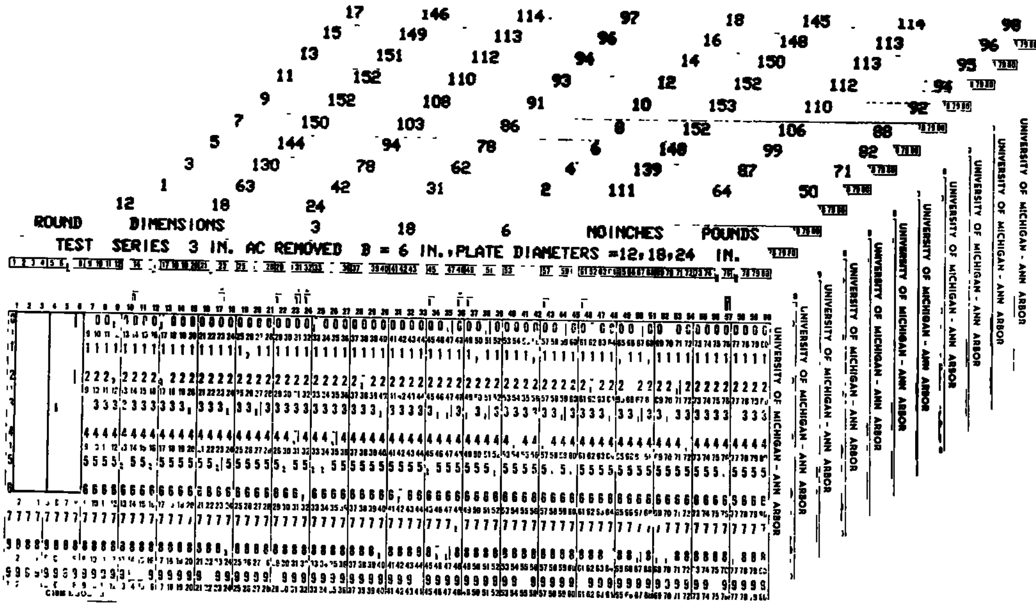


Figure 17. Example of input deck.

eries in the deck were for the same sizes of plates, the word "ROUND" could be re-laced by "SAME" which would prevent unnecessary duplication of computations al-eady carried out for the preceding test series.

Figure 18 shows a typical page of printed output. Although an example of the plot-ed output is not available, this system includes a plot-routine which is capable of reparing graphs and plotting results at the rate of 400 points in a full-page graph in bout 2.5 sec.

It is not intended to list the complete program here. It is felt, however, that some arts of the program should be reproduced to indicate how the MAD language and other imilar languages are being developed to make the use of digital computers more ac-essible to a person who is not in a position to spend the time and energy to study the etails of the internal functions of the computer. It may be said today that learning write programs in the MAD language (that is, learning to use the computer) is an-logous to learning to drive an automobile. One may perfect the former technique ithout acquiring much knowledge of computers themselves.

A very powerful statement in the MAD language is the "WHENEVER-Statement." o demonstrate this, reference is made to the input cards shown in Figure 17. De-nding on the first and second words on Card 2, it is possible to deduce the P/A tio in various ways. For round and square plates, this may be as follows:

```
WHENEVER SHAPE .E. $ SQUARE $. .AND. SIZE .E. $ AREAS $
PERARE (J) = 4. / SQRT. (TEMP (I))
OR WHENEVER SHAPE .E. $ ROUND $. .AND. SIZE .E. $ AREAS $
PERARE (J) = 2. / SQRT. (TEMP (I) / 3.14)
OTHERWISE
PERARE (J) = 4. / TEMP (I)
END OF CONDITIONAL
```

Most of the abbreviations used in the above sequence are self-explanatory. "TEMP

ANALYSIS OF PLATE BEARING TEST DATA

TEST SERIES 3 IN. AC REMOVED B = 6 IN., PLATE DIAMETERS = 12, 18, 24 IN.

SETTLEMENT DELTA	OBSERVED PRESSURE	COMPUTED PRESSURE	PERCENTAGE DIFFERENCE	PERIMETER SHEAR M	DEVELOPED PRESSURE N	M * P/A	K 1 DELTA/N	K 2 M/N
INCHES	P./SQ.I.	P./SQ.I.		P./I.	P./SQ.I.	P./SQ.I.	CU.I./P.	I.
0.1	63.00	63.07	-0.11	191.57	-0.79	63.86	-0.12727	-243.819
	42.00	41.79	0.51			42.57		
	31.00	31.14	-0.46			31.93		
0.2	111.00	109.64	1.22	374.14	-15.07	124.71	-0.01327	-24.825
	64.00	68.07	-6.36			83.14		
	50.00	47.29	5.43			62 36		
0.3	130.00	128.57	1.10	416.57	-10.29	138.86	-0.02917	-40.500
	78.00	82.29	-5.49			92.57		
	62.00	59.14	4.61			69.43		
0.4	139.00	137.57	1.03	416.57	-1.29	138.86	-0.31111	-324.002
	87.00	91.29	-4.93			92.57		
	71.00	68.14	4.02			69.43		
0.5	144.00	142.71	0.89	403.71	8.14	134.57	0.06140	49.579
	94.00	97.86	-4.10			89.71		
	78.00	75.43	3.30			67.29		
0.6	148.00	146.93	0.72	402.43	12.79	134.14	0.04693	31.475
	99.00	102.21	-3.25			89.43		
	82.00	79.86	2.63			67.07		
0.7	150.00	149.07	0.62	389.57	19.21	129.86	0.03643	20.275
	103.00	105.79	-2.70			86.57		
	86.00	84.14	2.16			64.93		

Figure 18. Example of printed output.

(I)" is a location in the memory of the computer where "AREAS" or "DIMENS" are stored. ".E." means "same as."

Another very interesting statement is the "THROUGH-Statement." An example of this follows:

```
    THROUGH D, FOR PLATE = 1,1, PLATE .G. PLNUMB
    SHEAR (SET, PLATE) = M (SET) PERARE (PLATE)
    COMPPR (SET, PLATE) = SHEAR (SET, PLATE) + N (SET)
    DIFFER (SET, PLATE) = (DEPRES (SET, PLATE + 1) - COMPPR
                          (SET, PLATE))
D   PERCT (SET, PLATE) = DIFFER (SET, PLATE) 100. / (DEPRES
                          (SET, PLATE + 1))
```

The first instruction would sound like this in plain English: "Go through all computations up to and including those in Line D; first, by putting the parameter "PLATE = 1," then, next time, by putting "PLATE = 1 + 1," and so on until "PLATE" is greater than "PLNUMB"."

The parameter "SET" stands for the deflection point being computed; that is, first, second, and so on. "M (SET)" and "N (SET)" are the constants m and n in Housel's linear equation. "COMPPR (SET, PLATE)" stands for computed pressure or bearing capacity, and "DEPRES (SET, PLATE + 1)" for observed bearing capacity. "(DEPRES (SET, 1))" stands for the amount of deflection, and "PLNUMB" is the number of plates used.

Any equality can be written in practically the same way one would when carrying out computations by hand. For example, if the stress coefficient K_1 referred to in this paper is to be computed, it is required only to add one instruction to the program.

$$K_1 \text{ (SET)} = \text{DEPRES (SET, 1) / N (SET)}$$

It should be clear from this that programming in MAD is not a very difficult task. Input and output instructions can, however, be tedious; but, by no means hard to understand.

The reader may be interested in getting an idea of the cost of carrying out the computations in this program.

Once the program has been written, the only requirement for processing data is to punch the data on cards, as shown in Figure 17. The punching is comparable to typewriting; hence, it would be difficult to give any definite figures as to how many cards one could expect to finish in a given time. This, however, would never be a very costly operation.

As an example of the cost of using the computer, it was found that the completion of 20 pages of output, as shown in Figure 18, took 1.6 min. The computer charges are $5,00 per min, and the foregoing would thus be about $8.00.

The time consumed in writing and testing the program itself was, in this case, the major factor. However, if it were found desirable to use it for substantial computations, the cost of programming would eventually be negligible.

One great advantage of the computer program is that it becomes easy and inexpensive to test out new theories and formulas which might be applicable to the program in question. Changes in the program itself are easy to make because instructions can be added or removed as required without changing the output and input to any great extent.

This example of the use of a high-speed digital computer has been included here for the reader who is not well acquainted with this powerful tool and who might be able to benefit from its use. It may be emphasized that it is not necessary to know the mechanical details of the computer itself to be able to use it, but merely to learn a relatively straightforward set of instructions such as those illustrated.

Discussion

W.S. HOUSEL, Professor of Civil Engineering, University of Michigan, and Research Consultant, Michigan State Highway Department—The writer has spent some time in an attempt to interpret the stress reactions developed in the Hybla Valley tests in the quantitative terms of the linear equation for bearing capacity used by the author, without coming to a final conclusion. This discussion will consequently be devoted to raising several questions yet to be answered and commenting on certain aspects of the structural behavior of flexible pavements.

Statistically, the linear equation reproduces the measured results of all the tests involved within a very narrow range of experimental error. Satisfying this test of validity does not reveal, in terms of structural behavior of the pavement structures, all of the factors which contribute to the surprisingly high values of perimeter shear, the inability of rigid plates to transmit direct pressure over the contact area, and the abnormally high deflections at which the full supporting capacity of the pavement structure is developed.

The fact that the maximum and minimum values of soil resistance coefficients derived from the linear equation for bearing capacity do determine the upper limit of static resistance or bearing capacity of the entire system has been demonstrated a number of times in the design of building foundations (1, 2). This relation has been confirmed in previous rigid plate bearing tests on flexible pavements (3, 4). If this principle is applied to the Hybla Valley tests, the limit of supporting capacity is not reached until the deflection is much higher than the range of thousandths of an inch normally considered in current practice. For example, in Figure 15 the critical deflection at a developed pressure of n = 0 is reached at approximately 0.8 in. for a total pavement thickness of 9 in. As shown in Figure 6, the same limits are not even reached in the Hybla Valley tests and would be at deflections considerably greater than 1 in.

Determining the source of these abnormally high deflections and correspondingly high values of perimeter shear is peculiarly perplexing. One may surmise that one possible source is in the permanent deformation due to yielding at the edges of the plate under the high pressure concentration along these edges. The increase in the critical deflection with increased thickness of base course suggests that consolidation or stress conditioning of the base courses is another potential source. Similar permanent deformation in the subgrade is another possible source that cannot be neglected. If the high deflections originate from these sources rather than in shearing displacement, it is important to recognize that the pavement structures will improve with time and load applications in service and that this greater range of available supporting capacity may eventually be mobilized. Either that or the sources of permanent deformation must be eliminated by greater initial compaction or the pavement must be designed with greater flexibility in order to develop this supporting capacity more effectively.

In this respect, current pavement design in this country may be penalizing itself by continued use of design criteria based on the elastic properties of rigid solids in which the assumed proportionality between total load and deflection takes precedence over th relationship between applied pressure and subgrade bearing capacity in plastic supporting media to which the linear equation for bearing capacity applies.

Rigidity and strength under the conditions of pavement performance are not synonymous. Rigidity carries with it susceptibility to fracture and the weakness of brittle failures. The objective of pavement design should be to build flexible strength or controlled flexibility into pavement structures. For most efficient performance, relative rigidity of the pavement components should be reduced to a minimum. Rigid pavemen surfaces should be made more flexible or the supporting elements of base and subgrad made more rigid. Flexible pavements have the advantage of mobilizing available subgrade support more effectively. There should be no prejudice against larger deflections as long as the yield value of the supporting subgrade or other pavement components is not exceeded and the structural continuity and riding quality of the pavement itself is not impaired.

This design philosophy calls for a rather definite reorientation of the current design practice which relies on proportionality between total load and deflection and relationships developed from the concept of a rigid pavement. It might be remarked tha one seldom sees steel wheels on a tea wagon; if there were, it might be as damaging to polished floors of hardwood and tile as the pinpoint heels of current ladies' shoes are to bituminous surfaces.

In this same connection, much of the difficulty with the analysis of rigid plate bear ing tests may be in their relative rigidity and the secondary dimensional effects whic they induce. These effects appear to mask the basic supporting capacity which the tests attempt to measure.

One method of eliminating this difficulty would be to make such tests with flexible bearing areas more nearly comparable to pneumatic tires. This procedure has been given some previous attention but has not yet supplanted the more common use of rig id plates adapted from foundation practice (5). Insofar as the writer is concerned, the attempt to unscramble the dimensional factors involved in perimeter shear and negative values of developed pressure has not been abandoned. There are some pron ising possibilities not completely explored, but any further progress in this directio must await further study.

In conclusion, it seems pertinent to make note of some European practices in pav ment design. By taking advantage of more liberal use of highly compacted granular subbases and structural continuity supplied by prestressing and hydraulic compressi units installed in the pavement base, surprising results are being obtained. In this connection, it has been reported that concrete pavements 3.5 to 7 in. thick are being generally built. One such pavement in Switzerland was reported to have been in service for several years under heavy traffic without having developed any cracks in some miles of pavement. These are practical accomplishments to which pavement designers in this country should be alert if they wish to keep abreast of the continued developments in pavement design.

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Comparative Studies of Combinations of Treated and Untreated Bases and Subbases for Flexible Pavements

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New Mexico's experimental Project No. F-051-1 (8) was constructed to compare "upside down" stabilization with other base construction. The term was applied to the design because it called for the subbase material to be treated with cement.

Nine experimental sections were constructed. The objective was to determine the effect of subbase stabilization compared to base course stabilization and the effect of a lower cement content in the base. Of particular interest is possible degradation of the mineral aggregates in all sections. The treated subbase sections should eliminate intrusion of subgrade soils into the base.

Through periodic inspections and check testing it is hoped that better knowledge can be obtained to determine which design provides the best protection for future distortion and roughness. An attempt will be made to evaluate the various designs relative to costs and serviceability.

● THROUGHOUT NEW MEXICO there has been a growing conviction that a subbase treated to obtain greater stability will solve many road construction problems. New Mexico's experimental Project F-051-1 (8) was constructed to compare "upside down" stabilization with other base construction. The term was applied to the design because it called for the subbase material to be treated with cement. The concept of building with great strength directly over weak subgrade soils reverses the accepted principle of building stability gradually upward for flexible base construction.

The basic design feature of placing untreated base materials over a rigid subbase was incorporated into several projects rebuilt in 1954. Several old concrete pavements in the vicinity of Albuquerque had become so cracked and distorted that reconstruction was necessary. The old pavements were covered with 6 in. of untreated base material compacted and reshaped to typical section. Over the reshaped section 2 in. of asphaltic hot plant mixed surfacing were placed. After six years of heavy traffic the surfaces remain in remarkably good condition. No reflective cracking has developed and string line checks show little rutting or distortion. Prior to 1954, old concrete pavements were overlayed with asphaltic mixtures. The pavements continu to pump under traffic, and distortion rapidly developed. Usually within a year the crack patterns of the old concrete reflected through the asphaltic surface.

In 1958, New Mexico commenced to use cement extensively to treat base course a gregates. Pattern cracking which appeared in the surface course caused much concern among road builders.

INTERSTATE 010-1 (8) 6, ROAD FORKS—EAST

On one New Mexico Project, I-010-1 (8) 6, Road Forks—East, the contractor became alarmed when, after having completed approximately one-half the length of the project, pattern cracking appeared in the plant mixed surface course. He requested permission to change his operations and process the cement in the subbase aggregat He pointed out good reasons for the change: immediate protection of the subgrade fr urface moisture, better compaction of the untreated base because of a firmer foun-lation, reflective cracking in the surface course alleviated by a cushioning interme-liate layer, and in all probability a smoother-riding road. In New Mexico practically .ll cement treatments are processed by road mix methods. The time specified to rocess, compact, and shape the treated materials did not permit the necessary blade vork to obtain the smoothness desired for surface course placement.

The New Mexico Highway Department had previously used variations of the upside own construction on urban projects where subgrade conditions were unfavorable to ood construction. Unstable subgrade soils caused by leaky water pipes and poor rainage were bridged by treating the subbase with cement. In all cases performance nder traffic appeared to be satisfactory. Because of the reasons stated by the con-ractor and the Department's previous experience, he was given permission to treat ne subbase instead of the base.

Without any planning or much forethought all the features of an experimental pro-ect were born. The contractor, in the interest of better flexible base construction, greed to construct other variations of base and subbase stabilization at no additional ost to the state. Variations paired were (a) untreated base and subbase; (b) base ourse treated with 1½ percent cement and subbase treated with 3 percent cement; nd (c) base course treated with 1½ percent cement placed over an untreated subbase. hroughout the project 3 in. of asphaltic plant mixed surfacing were laid, except for ne section of the interstate connection where 1½ in. of plant mix were placed over an ntreated base and a subbase treated with 3 percent cement.

PRELIMINARY DISCUSSIONS, F-051-1 (8)

The materials and testing laboratory recommended the upside down design for sev-al projects. One of the projects so recommended was located on US 64 north of .nta Fe, between Tesuque and Pojoaque. Samples taken from the subgrade soils ere found to be loaded with mica on which water acted rapidly and caused a greater ss of stability than normally expected for the soils encountered. It was thought that ment stabilization of the subbase would prevent any intrusion of the micaceous ma-rials into the base.

Bureau of Public Roads engineers pointed out that the limited use of the design did t provide enough background for standard application. Following normal procedure ey requested further justification and documentation before approval could be given r its use. Several conferences ensued and the facets of the design were discussed some detail.

The discussions disclosed opinions which differed on whether or not reflective acking was a forerunner of distress. Several engineers believed that cracking was desirable but thought it could be alleviated by reducing the amount of cement used. hers thought that cement would be of little benefit unless slab strength were devel-ed. Ideas about the upside down design centered on the untreated base course layer. e engineer felt strongly that the aggregates should be of top quality, well-graded, l the fines sandy and nonplastic. Samples tested from one of the Albuquerque pro-ts, reconstructed in 1954, had plastic indexes ranging from five to seven. The ne engineer pointed out that the dynamic forces from moving loads were more or s confined within a granular layer and could be causing degradation of the aggre-es which may have caused the material to be plastic. Project records showed ne plasticity, but the issue was not clear.

Another engineer introduced the subject of asphalt. He believed that asphaltic-ated materials would perform equally as well as cement-treated aggregates. Up-e down or right side up, reflective cracking would not be a problem. No one, so as is known, brought up the subject of lime. However, some conjecture developed ut the need of treating either base or subbase aggregates. Where was the proof any benefits existed? One thing was certain: Factual information supported by entific data were not available for many of the ideas expressed.

INFORMATION ABOUT EXPERIMENTAL SECTIONS

Eventually, treatment of the subbase with cement was chosen for the basic structural design of Project F-051-1 (8), but included were experimental sections each 2,000 ft long to make comparative studies of treated and untreated bases and subbases. The make-up of each experimental section was restricted to those discussed and about which the proponents seemed to have strong convictions. It might be said that the experimental Project F-051-1 (8) came about because of differences of opinion among engineers and a desire to know the truth.

It was agreed to construct each section to full stabilization, which in New Mexico is determined by the relationship between the traffic index and the California R. Values. Credit for gravel equivalent thickness of 1½ times was taken for both the asphalt and cement stabiliza-

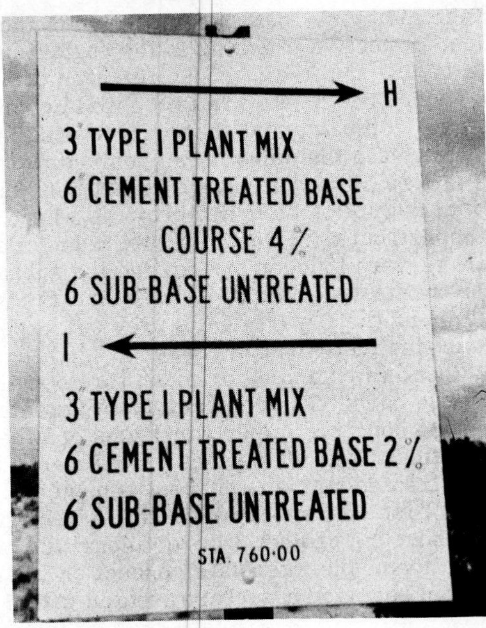


Figure 1. Information sign for Section H.

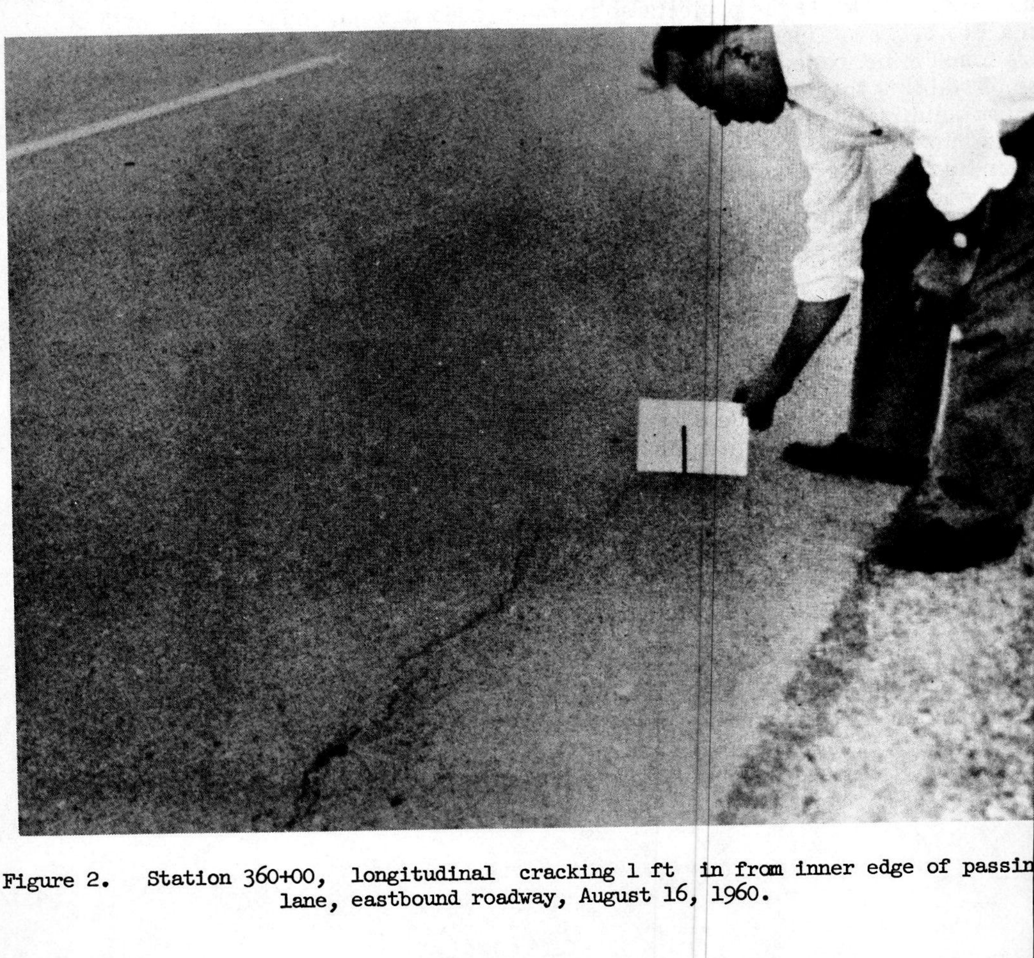


Figure 2. Station 360+00, longitudinal cracking 1 ft in from inner edge of passin lane, eastbound roadway, August 16, 1960.

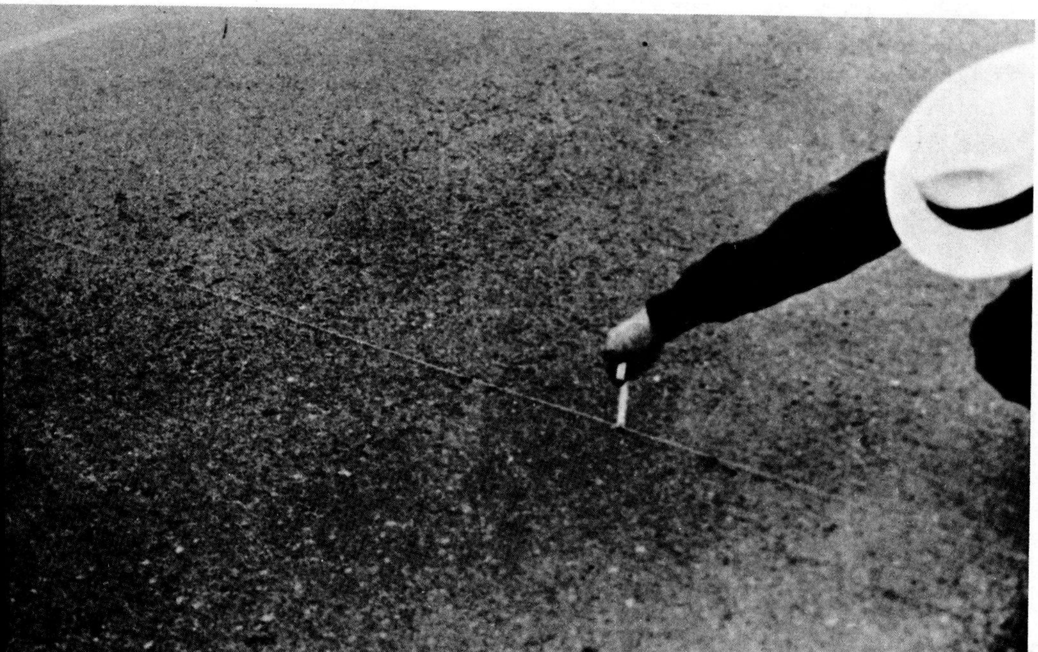
ion where 4 percent additives were used and for the asphaltic surface course. No redit was taken for the Class C stabilization in the section using 2 percent cement.

The same company which built I-010-1 (8) 6, Road Forks—East, was awarded the ontract. The company tried earnestly to comply with each letter of the specifica-ions. R. L. Baker, project engineer, supervised the work. John Jaramillo, labor-tory technician from the central laboratory, inspected the work, lifted the samples, nd compiled the records. All record samples were taken after the work was com-leted and tested in the central laboratory. The top 6 in. of subgrade, the subbase, nd the base courses were specified to be compacted to a minimum of 95 percent mod-ified Proctor density. Density tests of the completed work show that compactions vell above the minimum requirements were generally obtained.

Because of plastic and nonplastic requirements, two separate material pits were esignated for production of mineral aggregates for base, subbase, and surface con-truction. One was located in the Pojoaque River, from which the nonplastic base nd surface course materials were obtained. The other was from a hill deposit which ontained natural fines compatible to obtain plastic indexes ranging from three to six.

To assist inspection of this project there are signs at the beginning and end of each esign change with information giving the stations and how each section is constructed Fig. 1). There are nine experimental test sections designated by letters A, B, C, D, , F, G, H, I. Section A is the control section and is typical of both right and left anes throughout the project, excepting the comparative experimental group B through . All the comparative sections were constructed on the northbound lane.

The contractor's superintendent was asked which of the experimental sections he ad found the easiest to construct. He replied that he preferred either the asphalt-



re 3. Station 460+00, $\frac{1}{4}$-in. rutting in outer wheel path of traffic lane, east-bound roadway, August 16, 1960.

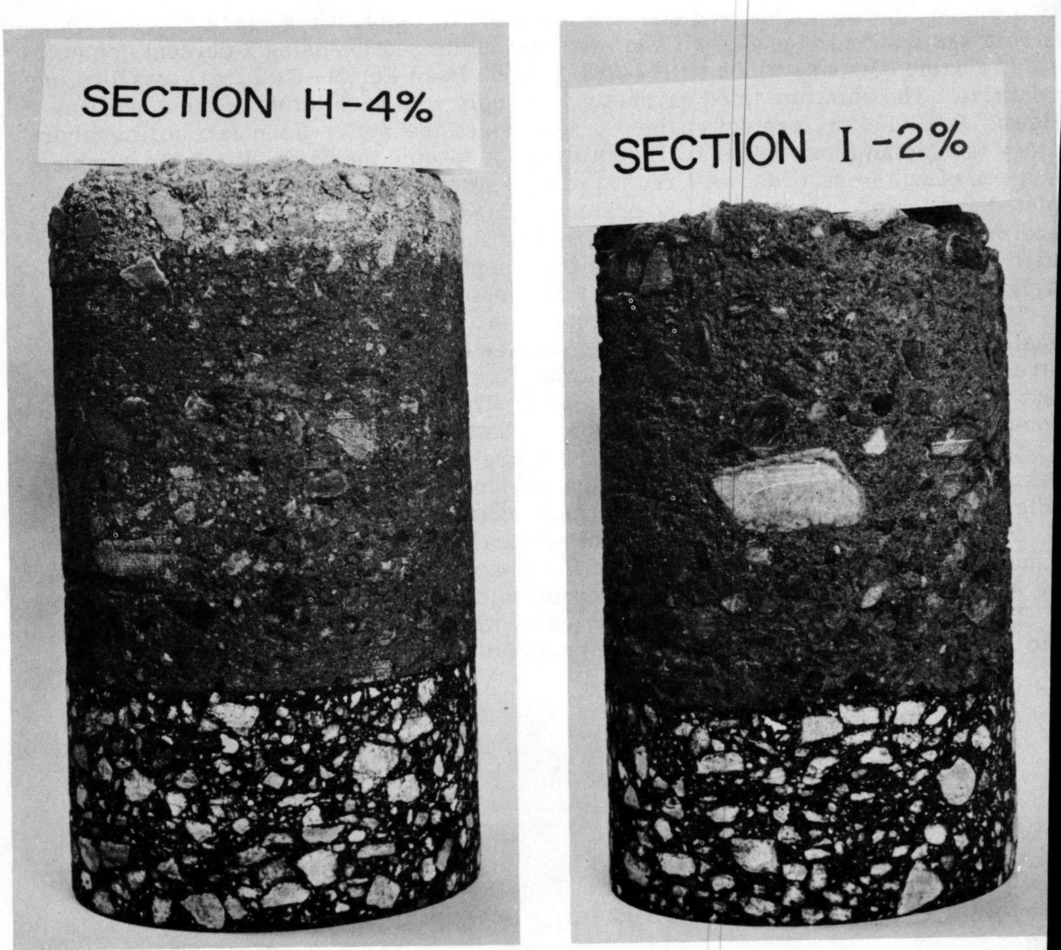


Figure 4. Cores taken from experimental project Sections H and I.

treated base or the upside down construction having a three to six plastic index in the intermediate layer. The sandy nonplastic material was difficult to hold to the typical section.

INSPECTION COMMENTS, F-051-1 (8)

On August 15, 1960, the first official examination of the completed experimental sections was made (Figs. 2 and 3). Observing the tests were W. L. Eager and L. H. Miller from the Bureau of Public Roads; and C. W. Johnson, and John J. Plese from the New Mexico State Highway Department.

Road roughnesses were measured with the Regional Bureau of Public Roads rough ness indicator through the experimental sections. It was desired to obtain initial roughness readings before any change had occurred through traffic or natural conditions. All of the sections gave good readings, although there is some indication that sections which have treated base course materials immediately under the mat are rougher than other sections. These results will be compared with future tests durin the life of the experimental work. Tabulation of the results obtained are attached to the Appendices of this paper.

String line checks were made on each section to determine if any rutting had deve oped from contractor's trucks hauling over the completed work. No rutting was four on any of the experimental sections on F-051-1 (8), Tesuque-Pojoaque.

The only surface cracks found were in Sections H and I, where the base was treated with cement immediately under the mat. Section H was treated with 4 percent cement and Section I was treated with 2 percent cement (Fig. 4). Transverse and pattern cracking were noted in both sections, but none were thought to be damaging as yet. The best indication of what to expect came from a previous survey of regularly-spaced transverse shoulder cracks where the plant mix was laid 1½ in. thick. One hundred and thirty-six transverse cracks were found in Section H, where 4 percent cement was used. One hundred and thirty-seven cracks were found in the shoulder of Section I, where 2 percent cement was used.

On November 10, 1960, Benkelman beam deflections were measured at three separate locations of each experimental section. Using 10,800-lb wheel loads the average results ranged from 14.4 to 24.0 thousandths of an inch, which was considered good. As could be expected, readings were higher for Sections E and F, where neither the base nor subbase were treated.

INSPECTION COMMENTS, I-010-1 (8) 6

After one year of heavy traffic, rutting in the surface had developed to a depth of ¼ in. on the Road Forks—East Project, I-010-1 (8) 6. No pronounced differences could be perceived in the upside down or conventional stabilizations. Longitudinal cracks about 1 ft from the paved shoulder were pronounced in the passing lane from station 326+15 to station 600+00, where the base was stabilized with 3 percent cement. From station 600+00 to station 800+00, where the subbase was treated with cement, the longitudinal cracks were located in the paved shoulder about 2 ft over, relative to the other crack position. Longitudinal cracks and rutting appear to be more associated with soil and moisture conditions than with the design of base and subbase courses. The road from station 326+15 to station 800+00 traverses a shallow lake with alternately dry and wet cycles (Fig. 5). Summer traffic seemed to have closed up most of the transverse reflective cracking from the cement-treated base. These cracks will no doubt tend to open up during colder weather. Roughness readings (tabulated in the Appendices) were somewhat rougher than the initial readings recorded on F-051-1 (8). Inasmuch as roughness measurements were not taken immediately after construction on I-010-1 (8) it is not known if traffic and weathering contribute to roughness.

Figure 5. Typical high shrinkage clay soil in bed of dry lake, August 16, 1960.

Information about design requirements and tests data covering compaction densities, roughness measurements, and Benkelman beam readings for both I-010-1 (8) 6 and F-051-1 (8), experimental projects is in the Appendices.

OBJECTIVES

The objectives of the comparative sections were to determine the effect of subbase stabilization and the effects of other design variations.

Through periodic inspections and check testing it is hoped that better knowledge can be obtained to determine which design provides the best protection from future distortion and roughness. Of particular interest is possible degradation of the mineral aggregates in all sections. It is felt that the treated subbase sections should eliminate intrusion of subgrade soils into the base and therefore provide a good opportunity to determine if degradation is actually taking place. Assuming that it does take place, it would be desirable to know the rate and amount of degradation that can be expected before distress in the surface is indicated. Because reflective cracking has provoked so much discussion, the Department hopes to determine if this defect contributes to distortion and roughness developing in the riding surface.

Although economy was not considered in the original planning, everyone is interested in contract and maintenance costs. An attempt will be made to evaluate the various designs relative to costs and serviceability in the hope that a guide can be established to determine which is the best bargain for the money expended.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the valuable assistance rendered by O.G. Betancourt, of the central laboratory, in the preparation of this paper.

Appendix A

F-058-1 (8) TESUQUE-POJOAQUE

EXPERIMENTAL PROJECT TEST SECTIONS

PROJECT F-051-1 (8) TESUQUE – POJOAQUE

B. O. P. STA. 387+96 E.O.P. STA. 819+00
Test sections begin at Sta. 600+00 and end at Sta. 780+00

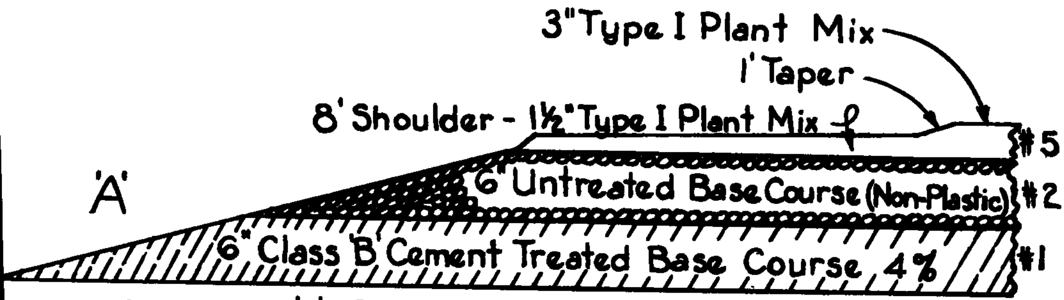
TEST SECTIONS: A, B, C, D, E, F, G, H, I.

Note: Section A is typical of both right and left lanes for the entire project, excluding test sections B through I.

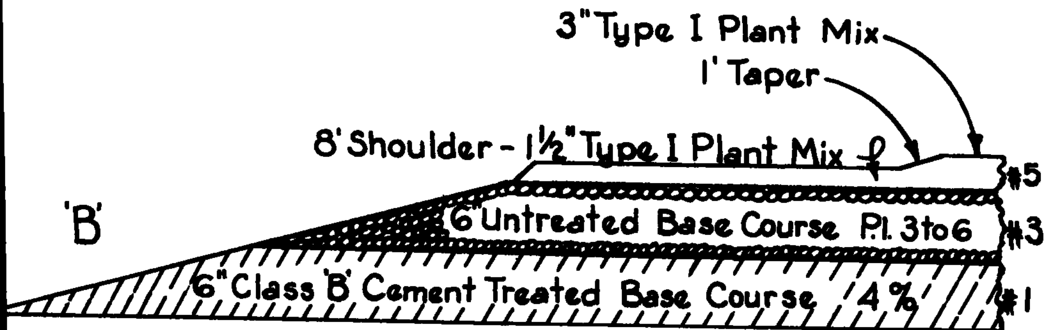
#1 - Cement-treated base course produced from Pit No. 58-126-S.
#2 - Untreated base course and asphalt-treated base course produced from Pit No. 58-124-S (non-plastic material)
#3 - Untreated base course with P.I. from 3 to 6 produced from Pit. No. 58-126-S.
#4 - Subbase controlled gradation produced from Pit No. 58-124-S and Pit No. 58-126-S.
#5)
#6) Plant mix and mineral aggregate for shoulder treatment produced from Pit No. 58-124-S.

RECOMMENDED SPECIFICATIONS FOR SURFACING AGGREGATES· % PASSING

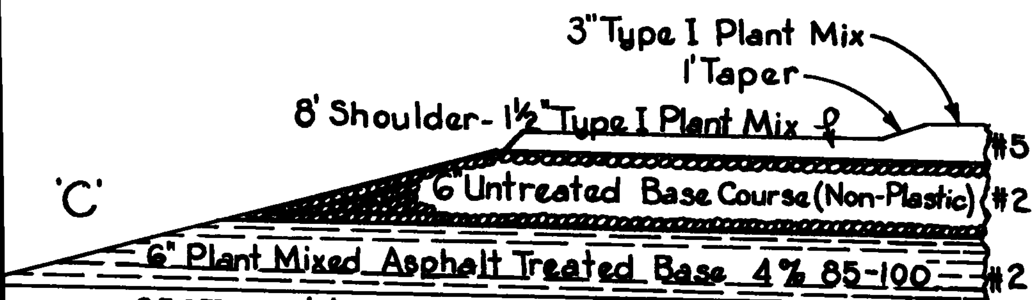
Sieve Size	#1 Base Course Cement Treated	#2 Base Course Untreated & Asphalt-Treated	#3 Base Course Untreated P.I 3 to 6	#4 Subbase Controlled Gradation	#5 Plant Mix Type I B	#6 Mineral Agg. Shoulder Treatment.
2"				100		
1"	100	100	100	70-100		
3/4"	85-100	80-100	80-100		100	
5/8"						100
1/2"					70-100	
3/8"					55-85	
No. 4	40-70	30-60	30-60	30-55	40-65	0-20
No. 10	30-55	20-45	20-45	20-40	30-50	0-4
No. 40					15-30	
No. 80					8-20	
No. 200	6-15	4-12	4-12	4-12	4-9	
L. L.	25 or less	Sandy	25 or less	35 or less	Sandy	
P. I.	6 or less	Non-Plastic	3 to 6	6 or less	Nonplastic	
L.A.Wear	50 or less	50 or less	50 or less	—	40 or less	40 or less



SECTION 'A' STA. 600+00 To 620+00 Rt. Lane

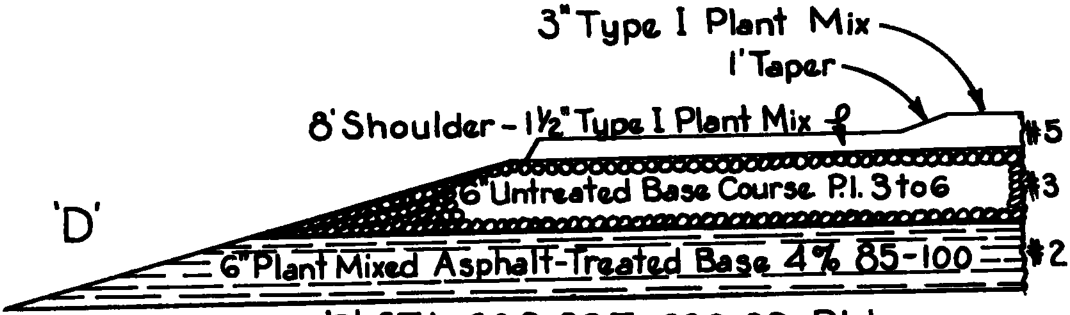


SECTION 'B' STA. 620+00 To 640+00 Rt. Lane



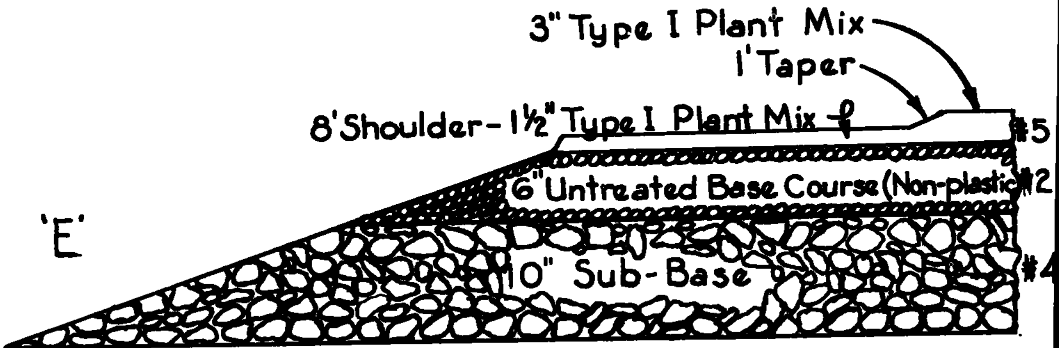
SECTION 'C' STA. 640+00 To 660+00 Rt. Lane

'D'



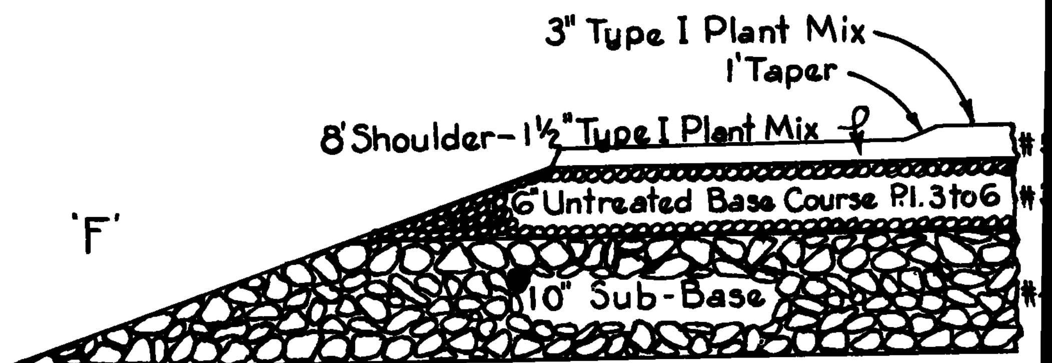
SECTION 'D' STA. 660+00 To 680+00 Rt. Lane

'E'

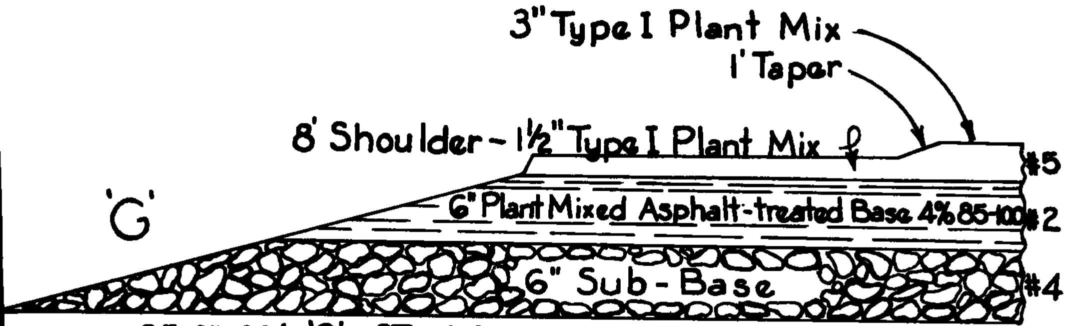


SECTION 'E' STA. 680+00 To 700+00 Rt. Lane

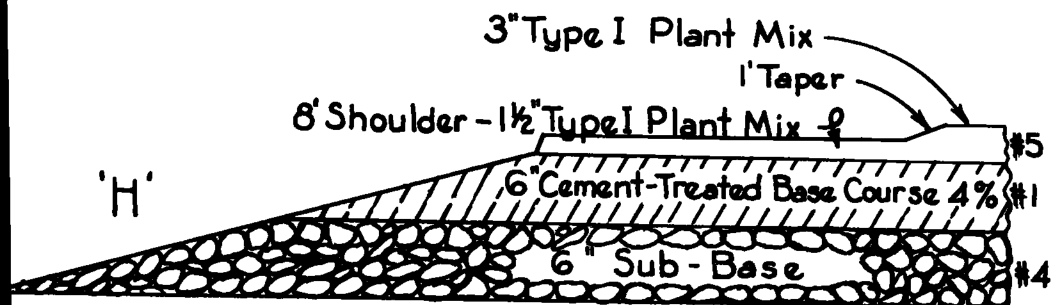
'F'



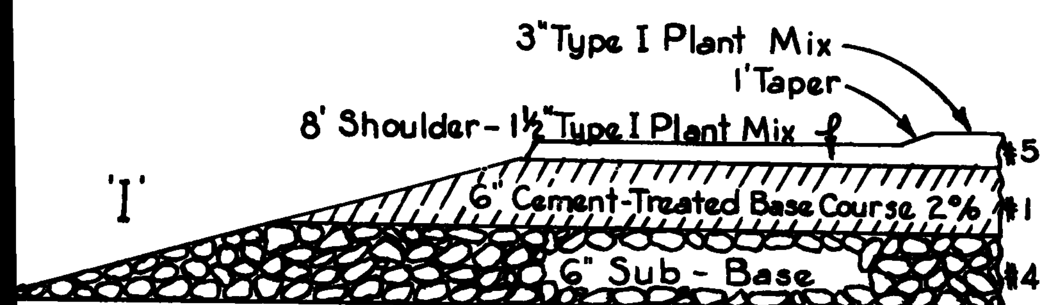
SECTION 'F' STA. 700+00 To 720+00 Rt. Lane



SECTION 'G' STA. 720+00 To 740+00 Rt. Lane



SECTION 'H' STA. 740+00 To 760+00 Rt. Lane



SECTION 'I' STA. 760+00 To 780+00 Rt. Lane

AVERAGE DENSITIES OBTAINED DURING CONSTRUCTION

New Mexico Project F-051-1 (8)

	MODIFIED PROCTOR				Plant Mixed Surface Course			
	Average Densities				% Theo. Density		% Lab. Density	
Section	Subgrade	Treated Base	Treated Base	Untreated Base	Bottom Course	Top Course	Bottom Course	Top Course
A	97.1	97.0[a]	-------	97.9	95.6	95.6	100.6	98.7
B	99.7	98.2[a]	-------	103.2	95.5	96.8	100.2	101.2
C	98.8	91.8[b]	100.5[c]	101.1	97.1	95.3	99.3	100.1
D	96.3	91.7[b]	99.2[c]	98.7	97.1	96.1	96.9	100.5
E	97.9	99.5[d]	-------	98.5	95.9	95.7	95.2	99.4
F	99.6	98.7[d]	-------	99.8	96.8	96.4	100.0	99.6
G	97.0	92.3[b]	99.2[c]	98.2[d]	96.6	95.4	99.98	98.2
H	96.4	99.5[d]	96.0[a]	-------	97.2	96.3	99.4	99.3
I	97.3	99.6[d]	96.5[a]	-------	97.6	95.7	98.6	99.3

[a]cement-treated base

[b]asphalt-treated base; % theo. density

[c]asphalt-treated base; % lab. density

[d]subbase

Subgrade, subbase, untreated base, and cement-treated base: modified proctor density. Asphalt-treated base and plant mixed surfacing: Marshall hammer, 75 blows on each side.

SUMMARY OF SURFACE ROUGHNESS MEASUREMENTS

New Mexico Project F-051-1(8)

Tesuque-Pojoaque

August 15, 1960

				Roughness		
				Going North (1)		Going So
Sect.	Station to Station	Subbase	Base	In/Sect.	In/Mi.	In/Sect.
A	600-620	6" CTB - 4%	6" Untreated No PI	16	42	18
B	620-640	6" CTB - 4%	6" Untreated 3-6 PI	18	47	20
C	640-660	6" ATB - 4%	6" Untreated No PI	18	47	20
D	660-680	6" ATB - 4%	6" Untreated 3-6 PI	18	47	20
E	680-700	10" Subbase (2")	6" Untreated No PI	21	55	19
F	700-720	10" Subbase (2")	6" Untreated 3-6 PI	19	50	23
G	720-740	6" Subbase (2")	6" ATB - 4%	21	55	24
H	740-760	6" Subbase (2")	6" CTB - 4%	23	61	24
I	760-780	6" Subbase (2")	6" CTB - 2%	21	55	21

NOTES:
3" Type One plant mix, 2 courses, on all sections
CTB = Cement Treated Base
ATB = Asphalt Treated Base
(1) = Outside or traffic lane
(2) = Inside or passing lane
Subbase = 2" maximum size, PI 6 or less

BENKELMAN BEAM TEST RESULTS
Project No. F-051-1(8)
Tesuque to Pojoaque

Date: 11-8-60 & 11-9-60
Wheel Load: L = 10810, R = 10800
Surface: 3" Plant Mix
Experimental Section: Sta. 600+00 to 780+00

All Tests Made in Driving Lane of North Bound Lane.

Station	Experimental Test Section	Deflection in Thousandth of an Inch Low	High	Average	Cut or fill section
601+00	A	8	12	10.4	Cut
610+00	A	12	18	16.4	Fill
617+00	A	12	24	16.6	Cut
622+00	B	18	22	19.3	Fill
625+75	B	16	22	19.7	Fill
635+00	B	14	22	18.8	Cut
643+00	C	16	22	19.0	Cut
650+50	C	16	20	17.3	Cut
657+74	C	12	16	14.3	Cut
663+00	D	12	16	14.0	Cut to fill
668+00	D	14	20	16.7	Cut
674+83	D	20	24	22.4	Cut
682+00	E	24	32	28.4	Cut
688+50	E	20	22	20.4	Fill
696+00	E	22	24	23.2	Grade
703+00	F	22	28	25.4	Fill
710+00	F	20	24	22.0	Fill
716+00	F	20	26	23.4	Fill
722+00	G	16	20	17.0	Cut
730+50	G	18	20	19.6	Fill
736+11	G	14	16	15.6	Fill
742+84	H	14	22	19.7	Fill
749+25	H	16	20	17.6	Cut
757+00	H	6	10	7.0	Cut
763+60	I	12	16	14.2	Cut
772+50	I	12	14	13.0	Cut
778+44	I	22	26	24.0	Fill

Appendix B

I-010-1 (8) 6 ROAD FORKS—EAST

EXPERIMENTAL PROJECT TEST SECTIONS

PROJECT I-010-1 (8)6 ROAD FORKS - EAST

B.O.P. STA. 326+15.47 E.O.P. STA. 1088+28.4

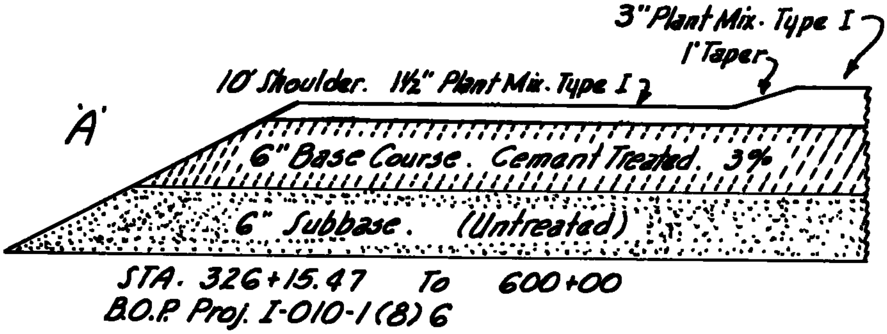
TEST SECTIONS A, B, C, D, E, F, G, H.

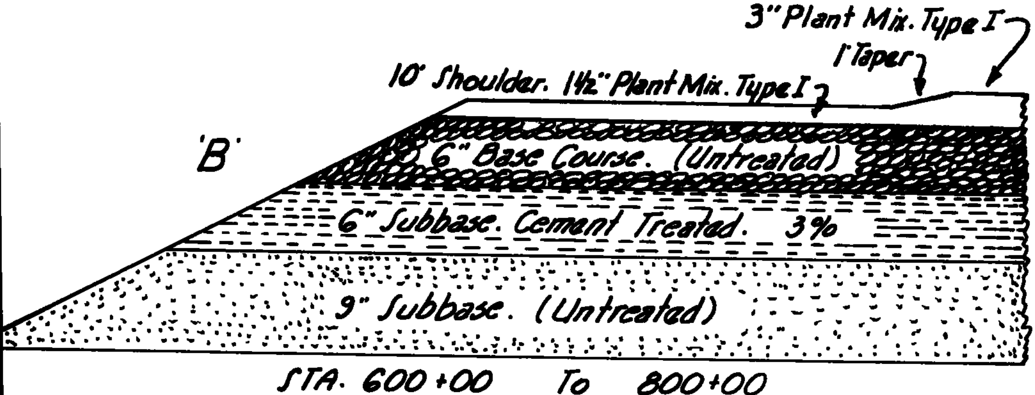
Subbase Material produced from Pit No. 58-29-S.

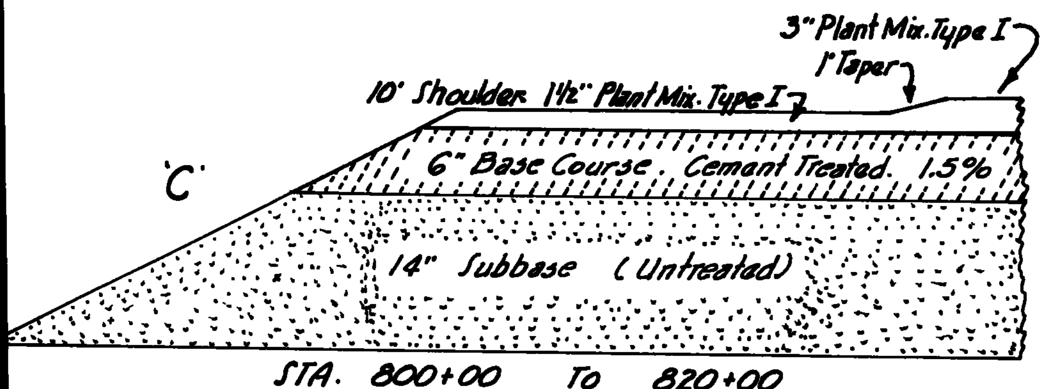
Base course, plant mix, and surface treatment aggregate produced from Pit No. 58-G2-S.

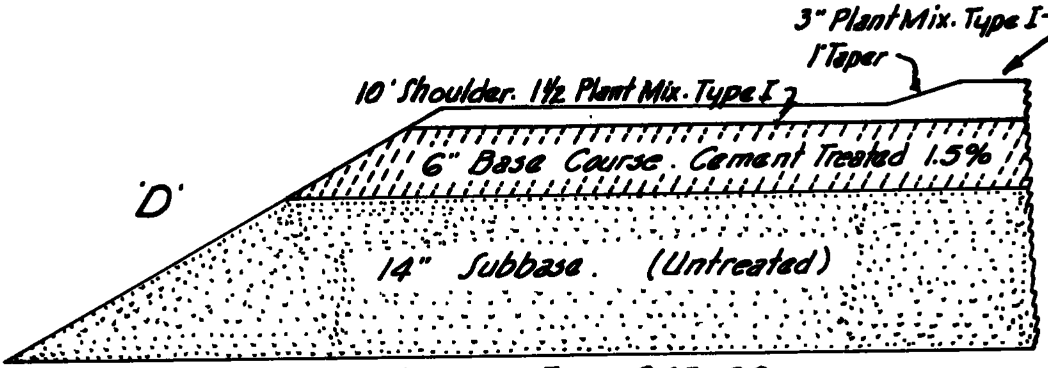
RECOMMENDED SPECIFICATIONS FOR SURFACING AGGREGATES: % PASSING

Sieve Size	Subbase Controlled Gradation	Base Course	Mineral Agg. Plant Mix Type I	Mineral Agg. Surface Treat.	Mineral Aggregate Surface Treatment Connectio 1st. Course	Mineral Aggregate Surface Treatment Connectio 2nd Cours
2"	100					
1"		100				
3/4"		80 - 100	100		100	
5/8"				100		
1/2"			75 - 100			100
3/8"			67 - 85		0 - 25	
No. 4	25 - 70	30 - 60	50 - 65	0 - 20		0 - 20
No. 10	20 - 55	20 - 45	34 - 47	0 - 4	0 - 4	0 - 4
No. 40			14 - 24			
No. 80			8 - 16			
No. 200	4 - 15	4 - 12	4 - 8			
L.L.	35 or less	25 or less	Sandy			
P.I.	6 or less	6 or less	Non Plastic			
L.A.Wear	–	50 or less	50 or less	40 or less	40 or less	40 or l

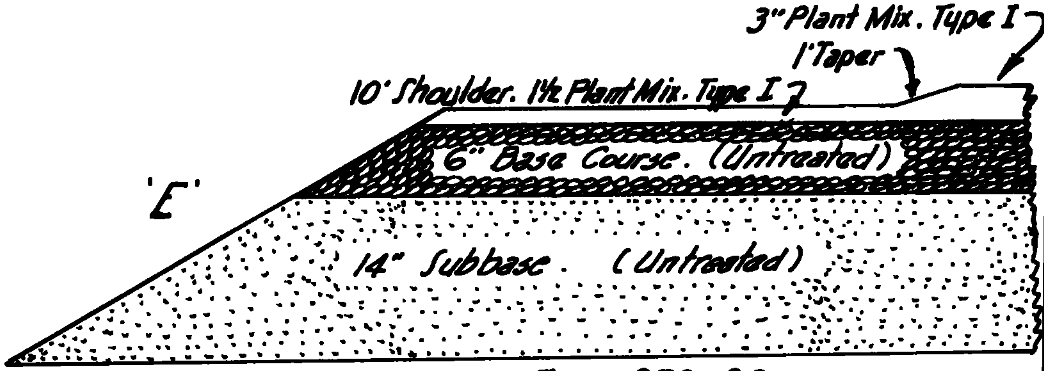

3" Plant Mix. Type I
1" Taper
10' Shoulder. 1½" Plant Mix. Type I
'A'
6" Base Course. Cemant Treated. 3%
6" Subbase. (Untreated)
STA. 326+15.47 To 600+00
B.O.P. Proj. I-010-1(8)6


3" Plant Mix. Type I
1" Taper
10' Shoulder. 1½" Plant Mix. Type I
'B'
6" Base Course. (Untreated)
6" Subbase. Cement Treated. 3%
9" Subbase. (Untreated)
STA. 600+00 To 800+00

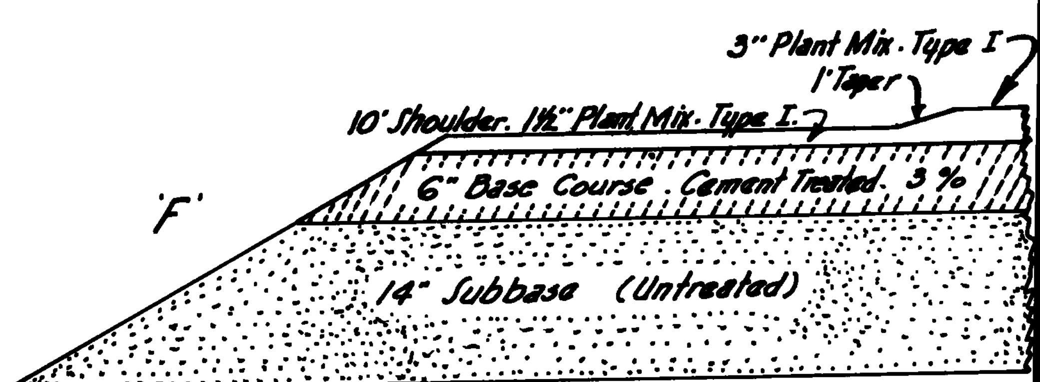

3" Plant Mix. Type I
1" Taper
10' Shoulder. 1½" Plant Mix. Type I
'C'
6" Base Course. Cemant Treated. 1.5%
14" Subbase (Untreated)
STA. 800+00 To 820+00



STA. 820+00 To 845+00

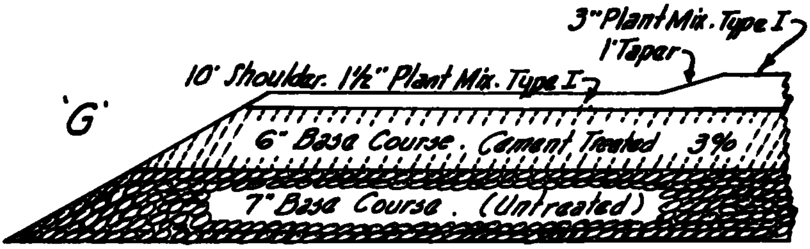


STA. 845+00 To 870+00



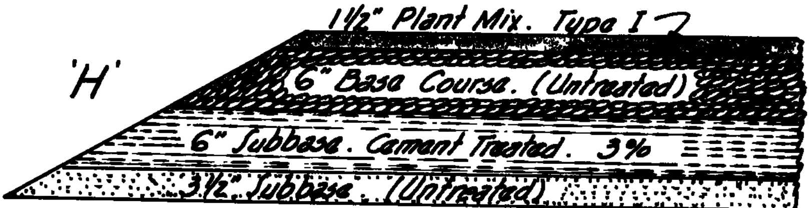
STA. 870+00 To 990+00

'G'



STA. 990+00 To 1036+54
E.O.P. I-010-1(8)6

'H'



STA. 1036+54 To 1069+07
Begin Connection. End Connection.

CONDITION SURVEY
New Mexico I-010-1(8)6
Road Forks - East
August 16, 1960

STATION TO STATION	SUBBASE	BASE	RUTTING [a]	Cracking [b] TRANSVERSE	Cracking [b] LONGITUDINAL	ROUGHN INCH/M
326+154 to 600	6" Untreated	6" CTB-3%	1/4"	Inner Edge & Shoulders	Inner Edge & Shoulders	64.6
600 to 800	9" Untreated 6" CT - 3%	6" Untreated	3/16"	None	Some Shoulders	62.7
800 to 820	8" Untreated 6" CT - 3%	6" CTB - 1½%	1/8"	None	None	66 2
820 to 845	14" Untreated	6" CTB - 1½%	1/4"	None	None	59.
845 to 870	14" Untreated	6" Untreated	1/4"	None	None	63.
870 to 990	14" Untreated	6" CTB - 3%	3/16"	None	None	68.
990 to 1036+54	7" Untreated	6" CTB - 3%	1/8"	None	None	66.
1036+54 to 1069+07 [c]	3½" Untreated 6" CT - 3%	6" Untreated	1/8"	None	None	78.
1069+07 to 1088+28	2" Untreated 6" CT - 3%	6" Untreated	1/8"	None	None	

a - In outer wheel path - traffic lane.

b - Where cracking marked "none" indicates could not be observed at this time - might be evident in cold weather.

c - 1½" plant mix mat - 3" 2-course plant mix all other sections

BENKELMAN BEAM TEST RESULTS
N. M. Project No. I-010-1 (8) 6,
Road Forks - East

DATE: 11-29-60
Wheel Load L = 10810, R-10800

Station	Experimental Test Section	Deflection in Thousandths of an Inch Low	High	Average	Cut or Fill	Experimental Sections
350+00	A	8	18	13.6	Fill	Lake Bed
390+00	A	24	30	26.8	Fill	
440+00	A	20	26	22.4	Fill	
490+00	A	12	16	14.8	Fill	
560+00	A	14	30	19.6	Fill	
260+00	B	14	22	18.4	Fill	
660+00	B	14	18	16.3	Fill	
700+00	B	18	22	20.0	Fill	
740+00	B	12	16	15.2	Fill	
797+00	B	6	14	10.7	Fill	
805+00	C	6	16	12.8	Grade	Slope
810+00	C	10	14	11.7	Grade	
815+00	C	8	10	8.7	Grade	
825+00	D	12	18	15.0	Grade	
832+00	D	12	20	16.7	Grade	
840+00	D	10	18	14.7	Cut	Rolling Hills
850+00	E	14	18	16.4	Grade	
857+00	E	16	24	20.6	Cut	
865+00	E	18	20	18.0	Fill	
885+00	F	6	10	8.3	Cut	
900+00	F	10	12	11.3	Cut	
951+00	F	10	18	13.2	Grade	
985+00	F	4	8	6.8	Fill	
1005+00	-	8	14	10.0	Fill	
1020+00	-	8	14	12.3	Cut	
1035+00	-	10	14	11.6	Cut	
1045+00	-	18	22	20.0	Cut	
1055+00	-	14	20	17.3	Grade	
1065+50	-	12	18	14.8	Grade	
1074+00	-	10	14	11.6	Grade	
1079+00	-	14	18	16.0	Grade	
1084+00	-	12	16	13.7	Grade	

Plate Bearing Tests and Flexible Pavement Design in Florida

W. H. ZIMPFER, Associate Professor of Civil Engineering, University of Florida, Gainesville

● FIELD PLATE BEARING TESTS have been performed since 1958 in conjunction with the flexible pavement design research program sponsored by the Florida State Road Department. The first field tests were run using the 3-sq in. (1.95-in. diamete piston of the California Bearing Ratio test. This was followed by the use of 4-, 6-, 8-, 10- and 12-in. diameter rigid plate tests. The tests were initiated to obtain the bearing values of highway base, subbase and subgrade materials as separate layers and as composite pavement sections. All bearing values were related to the deflectio of the plate and the corresponding pressure on the plate. Recent tests have dealt with the bearing value of composite sections, including an asphalt concrete wearing surfac

The review of plate bearing tests, performed in the State of Florida, has been sub- divided into sections that are directly related to the various phases of the research pr gram, including (a) plate size and zone of stress, (b) variation of bearing values, (single layer relationships, (d) subgrade modulus as related to plate size, (e) two-la theory relationships, (f) thickness of wearing surface, and (g) repetitional loads.

PLATE SIZE AND ZONE OF STRESS

When a circular plate is loaded with a uniform load a zone beneath the plate is stressed. For a homogeneous semi-infinite mass, vertical stresses and maximum shearing stresses may be readily calculated by the use of equations developed by Jür- genson (1), Love and others. Of particular interest, when investigating the stresses associated with plate bearing tests on fle ible pavement layers, is the depth of the zone of significant stress as related to t diameter of the loaded area. The stress zone is often defined by a "pressure bull which defines points of equal stress inte sity. Accurate pressure bulb or contou of stress diagrams may be found in man publications. Some excellent diagrams appear in HRB Bulletin 114.

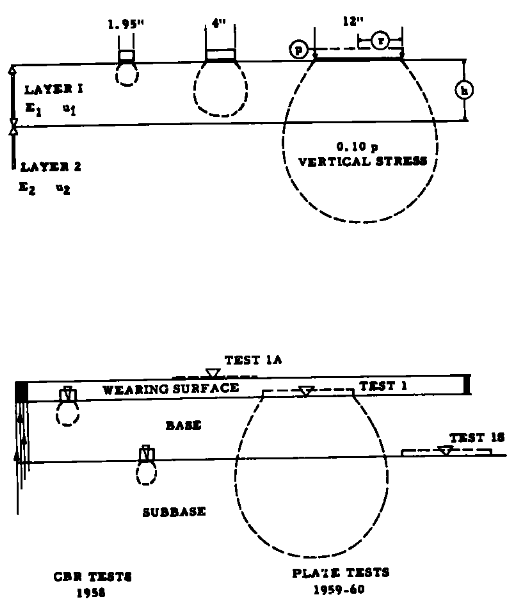


Figure 1. Plate size and pressure bulbs for plates and location of plate tests.

Figure 1a presents the pressure bulb corresponding to a vertical stress inten ty of 0.1 p for plates of 1.95-, 4-, and in. diameter. The depth of significant stress is about 1¾ times the diameter. For the maximum shearing stress of 0. p the depth of significant stress is abou 1¼ times the diameter. As can be seen the stressed zone increases in depth as the plate diameter increases. The CBI piston used for the original bearing test (1958) on base materials has a diamete of 1.95 in. Considering the stressed z under the piston it is obvious that the b ing value obtained is only a direct inde the strength of the base layer. Figure 1a also shows the stressed zone of the 12-in. diameter plate. It can be seen that for plate tests performed on the top of the base that a homogeneous mass of one layer does not exist throughout the stressed zone but a system of two layers is stressed. This system cannot be analyzed as a single layer but should be investigated as a layered system as was done by Burmister (2). Burmister's work is discussed later.

The effects of using a 1.95- and 12-in. diameter plate when testing a typical flexible pavement section are shown in Figure 1b. The advantages and limitations of each size plate are directly related to the depth and extent of the stressed zone. The small plate will give stress and displacement values of distinct and separate layers, whereas the large plate will give values of the layered section. Realizing most wheel loads have contact pressure areas that may be assumed circular, the use of the test data and theoretical stress computations for circular bearing areas may be used extensively for analysis.

TABLE 1

PERCENTAGE VARIATION OF BEARING VALUES; BASE STUDY, 1958

Material	Average of Maximum Values: 1.95-In. Plate (CBR)	4-In. Plate	8-In. Plate	12-In. Plate	Max. Values: 1.95-In. Plate (CBR)
(a) Standard CBR Tests					
nd clay	10	-	-	-	17
merock (N)	15	-	-	-	25
merock (S)	20	-	-	-	47
ab. shell	20	-	-	-	37
ell	25	-	-	-	49
(b) Load Increment Tests, ASTM 1196-57					
ay sand (4)	35	25	20	15	35

The early studies conducted in the state were with the 1.95-in. piston. The bearing sts were run on all typical base materials throughout the state and on most subbase l subgrade materials. Results were presented in reports (3) issued in 1958. These rly tests established the strength characteristics of the individual layers and later, connection with other test data, led to the development of a modified CBR design thod. This was possible because if the properties of the distinct layers are known l/or specified for field construction, a system of layers may be proportioned empirlly which will have a known field performance. Later tests utilized 8- and 12-in. meter plates to develop the relationships of layered systems which were and are be- investigated experimentally and theoretically.

VARIATION OF BEARING VALUES

The use of small plates has been investigated and was reported (4), in 1959. Conerable economy could be realized by performing tests with small diameter (1.95-in.) tes; however, small plates tend to give erratic and somewhat inconsistent results n performing duplicate tests. Small plates are more sensitive to soil variations in nogenity, large particles, and to surface conditions.

The base study, noted previously, included data which is directly related to the varon of bearing values. The results of tests, repeated a minimum of three times, led he development of Table 1 which gives the average of the maximum percentage varin and the maximum percentage variation for the tests reported in 1958.

Collins (4) gives an indication of the maximum percentage variation of the 1.95-in. plate and, in addition, the variation of the plates of larger sizes may be estimated from the data. These data are also given in Table 1. The effect of larger plates in reducing the percentage variation is evident.

TABLE 2

NUMBER OF TESTS REQUIRED TO GIVE A MEAN WITHIN 10 PERCENT OF TRUE MEAN WITH 95 PERCENT CERTAINTY

	Number of Tests					
Material	CBR Plate	3-In. Plate	4-In. Plate	6-In. Plate	10-In. Plate	12-In Plate
	(a) Plates loaded rapidly (5)					
Clay	18	4	-	10	-	-
Silty clay	34	3	-	26	-	-
Sand	42	14	-	9	-	-
Grav. sand	9	10	-	9	-	-
	(b) Load Increment Tests, ASTM 1196-57 (4)					
Clay sand	32	-	10	-	10	8

The percentage variation varied with soil type. This is expected because, as men- tioned previously, the scatter would be related to homogeneity, particle size and sur- face irregularities.

An attempt was made to analyze the data of Collins (4) using statistical methods. Sufficient data were not available for a reliable analysis; however, it is of interest t compare some preliminary calculations with those of Robinson and Lewis (5) who re- ported the results of a series of tests where 20 repetitions of each test were made to establish a true mean. The results of the study are given in Table 2. It may be note that no definite curve exists relating required number of tests and plate size but that a trend does appear. The number of tests required for the 6-in. plate is significantl less than the number required for the 1.95-in. plate. The 3-in. plate test results ar exceptional.

Using some of the data obtained in 1959, with a maximum of six repetitions of eac test, the number of tests required for identical criteria are noted in Table 2. Many additional repetitive tests are necessary in this area of study to establish relationshi between plate size, number of tests required, and soil type.

SINGLE-LAYER RELATIONSHIPS

A review of the single-layer theory as related to stress and deflection beneath a circular rigid plate was presented in previous reports (4, 6). The original problem of computing the stresses beneath a circular plate was solved by Boussinesq. Bous- sinesq obtained an equation for the deflection of a rigid plate located on a semi-infini elastic body as follows:

$$w = \frac{\pi \; pr}{2E} (1 - \mu^2)$$

in which

w = deflection
π = 3.14
p = pressure
r = radius of plate

E = modulus of elasticity
μ = Poisson's ratio
for $\mu = 0.5$
$w = 1.18 \frac{pr}{E}$ .... (average deflection of a rigid plate) for a flexible plate

$$w = 1.5 \frac{pr}{E}$$

'erzaghi, 1943, noted that soils were not truly elastic, but did retain the concept of lasticity and essentially replaced E by a soil modulus, M, which was equal to Mo + z. The resulting deflection equation may be written as

$$w = K' \frac{pr}{Mo + az}$$

/hen a = o and if w is constant, the pressure required to produce a given settlement ' is

$$p = \frac{w' \, Mo}{K' \, r} = K \frac{1}{r}$$

which

$$K = \frac{w' \, Mo}{K'}$$

ie foregoing equation is that of a hyperbola. The equation $p = K \frac{1}{r}$ is a theoretical relationship between pressure for a given w and plate size. If the soil modulus, M, is ried, a family of curves may be constructed.

SUBGRADE MODULUS AS RELATED TO PLATE SIZE

Reference 4 presented a set of curves developed from experimental data (Fig. 2), lating subgrade modulus, k, and diameter for various soil types. Noting that k is ual to the pressure at a given deflection divided by the deflection, it is possible to perimpose some theoretical curves of the $p = K (\frac{1}{r})$ type on the experimental data ig. 2). The theoretical and test curves show good agreement for a = o. Three different soil modulus values have been plotted to present a typical family of curves.

The relationship between subgrade modulus and plate size may be expressed in many ys in mathematical form. Because the relationship between CBR (Load Increment st) and larger plate sizes is of primary interest in Florida and is essentially one of bgrade modulus and plate size, the following equation is presented. CBR, plate size i pressure are related by:

$$p = K \frac{1}{r} \tag{1}$$

When w = 0.1 in., where 0.1 in. is the deflection of Standard CBR, the "CBR" Etion (Load Increment Test) becomes

$$p = \frac{10(CBR)}{r} \tag{2}$$

vhich

p = pressure, in psi;
CBR = ratio at 0.10-in. penetration; and
r = radius, in inches.

2 may be used to relate CBR, p, and r until additional test data are available. sonable agreement exists between test and theory for CBR values greater than 10. itional testing is necessary to relate the results of the Standard CBR Test and the d Increment Test (ASTM, 1196-57 and (4)).

TWO-LAYER THEORY RELATIONSHIPS

The analysis of the two-layer system was developed and presented by Burmister in 1943 and most recently discussed in 1958 (7). An investigation related to the two-layered system was conducted in 1960 and reported (6). Before discussing the results of the recent tests some comments about the layered system may be desirable to visualize the action of a typical system. A typical two-layer system is shown in Figure 1a. This system represents much closer agreement to the actual problem that exists when pavement sections are loaded either by wheel loads or plates. The effectiveness of spreading load or reducing vertical stress, when a reinforcing layer with a modulus E_1 is used over a second layer with a modulus E_2 less than E_1, has been discussed and illustrated by Burmister (7). The reduction is significant and the effectiveness of reduction increases as E_1/E_2 increases. Another factor of importance in the two-layer system is that of an increase of vertical stress gradient toward the interface, which in turn causes a shearing stress buildup. The shearing stress, as mentioned by Burmister, is much more important than in the Boussinesq case and must be sustained at the interface for continuity between the layers. Shearing stress could lead to failure

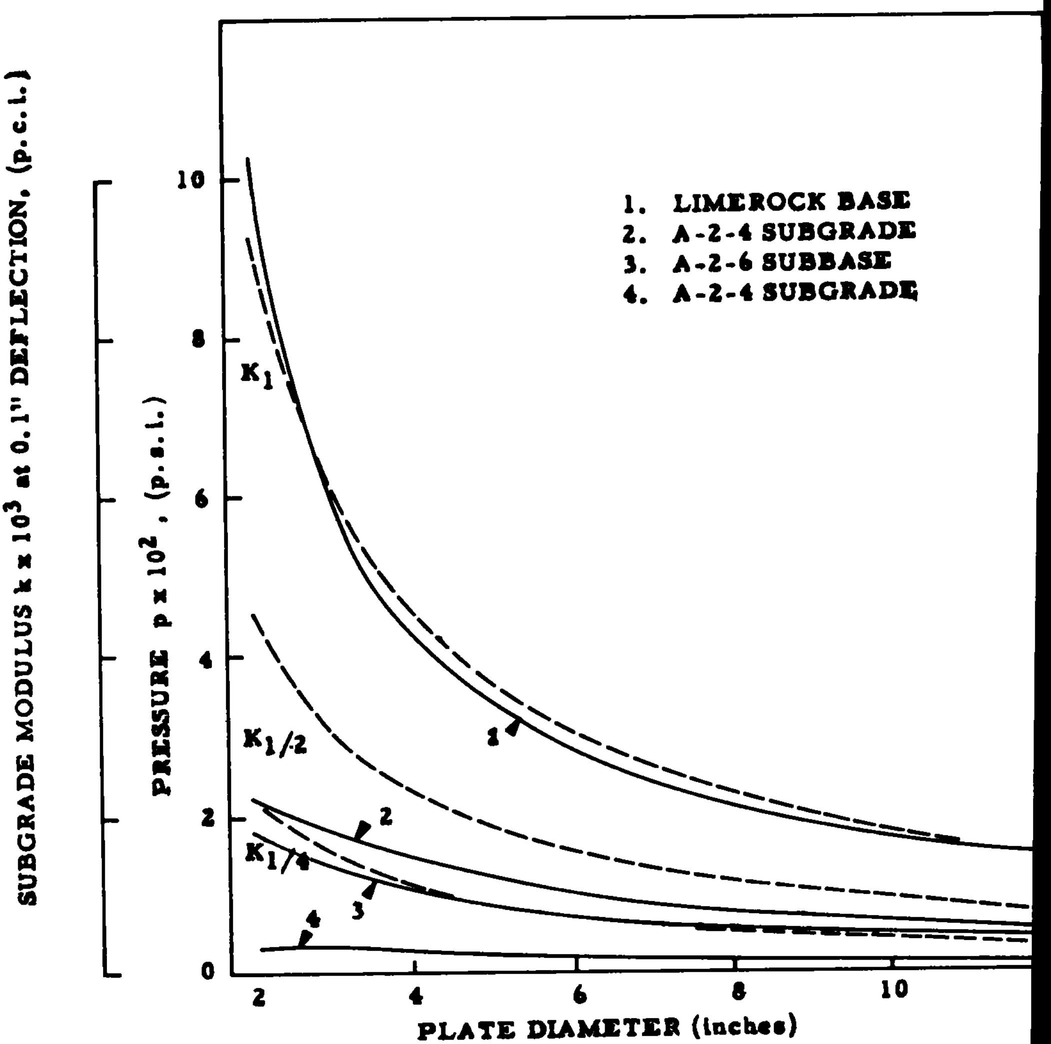


Figure 2. Subgrade modulus-diameter curves for some soils and theoretical curves.

due to excessive shearing strain; therefore, the deflection at the surface must be limited so that the shear stresses and strains are not critical. This limiting deflection may be about 0.05 in. for high-type flexible pavements constructed in Florida.

The deflection of a layered system as related to vertical stress and shear stress may be summarized by Burmister's influence curves of settlement coefficient, F_W (Fig. 3). The deflection equation for the layered system rigid plate is:

$$w = 1.18 \frac{pr}{E_2} \quad F_W \left[\frac{r}{h}, \frac{E_2}{E_1}\right] \tag{3}$$

or

$$w = 1.18 \frac{pr}{E_2} \quad F_W \tag{4}$$

Eq. 4 is in the same form as the Boussinesq equation for one layer and reduces to this ase for a one-layer system. In the two-layer system the settlement coefficient curves re related to r, h and E_2/E_1. The effect of these variables will be shown by the curves, oth test and theoretical, that follow.

The state conducted numerous tests on layered systems consisting of a typical Ocala imerock base material and a clay-sand subbase. The base thickness was varied from to 11 in. in controlled sections over a 600-ft test area, to study the effect of the ickness, h, of the reinforcing layer. Plates having diameters of 1.95, 4, 8, and 12 n. were used on the different base course thickness to study the effect of radius of late r and thickness of layer h. The results of these tests are presented in some deil in an earlier report (6).

Figure 4 shows the equipment used for some of the field testing, performed in connction with the recent plate bearing test studies.

The data obtained in recent studies have been re-evaluated and important parts are ımmarized and discussed.

To calculate the theoretical deflection, w, of the layered system, accurate values the moduli, E_2 and E_1, are necessary. It has been found (8) that a minimum of thickss of soil at least 1.5 times the diameter of the loaded plate is necessary for calcula-

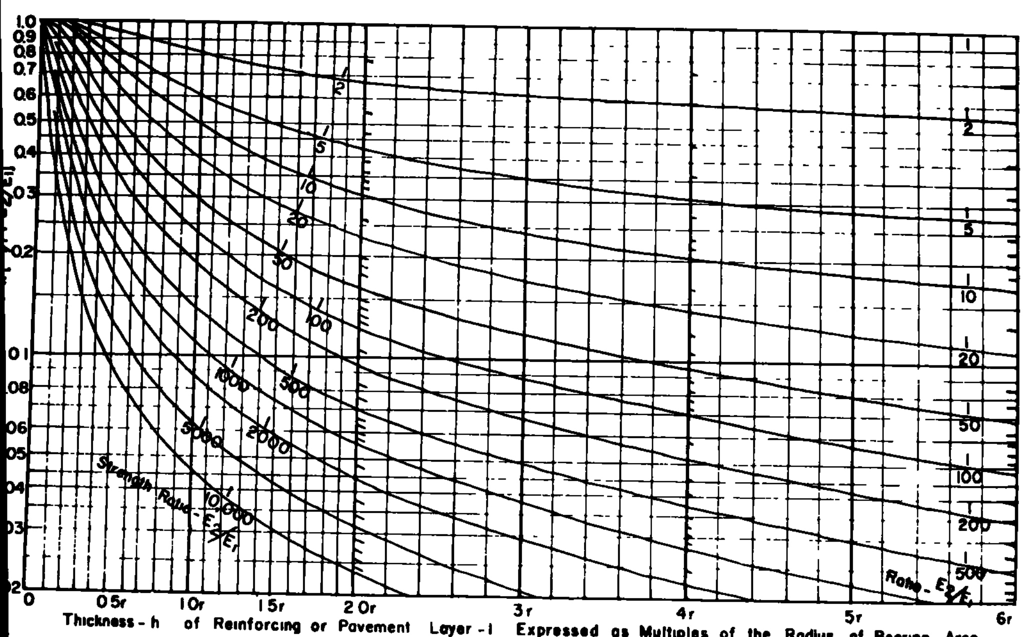


Figure 3. Influence curves of the settlement coefficient F_W (7).

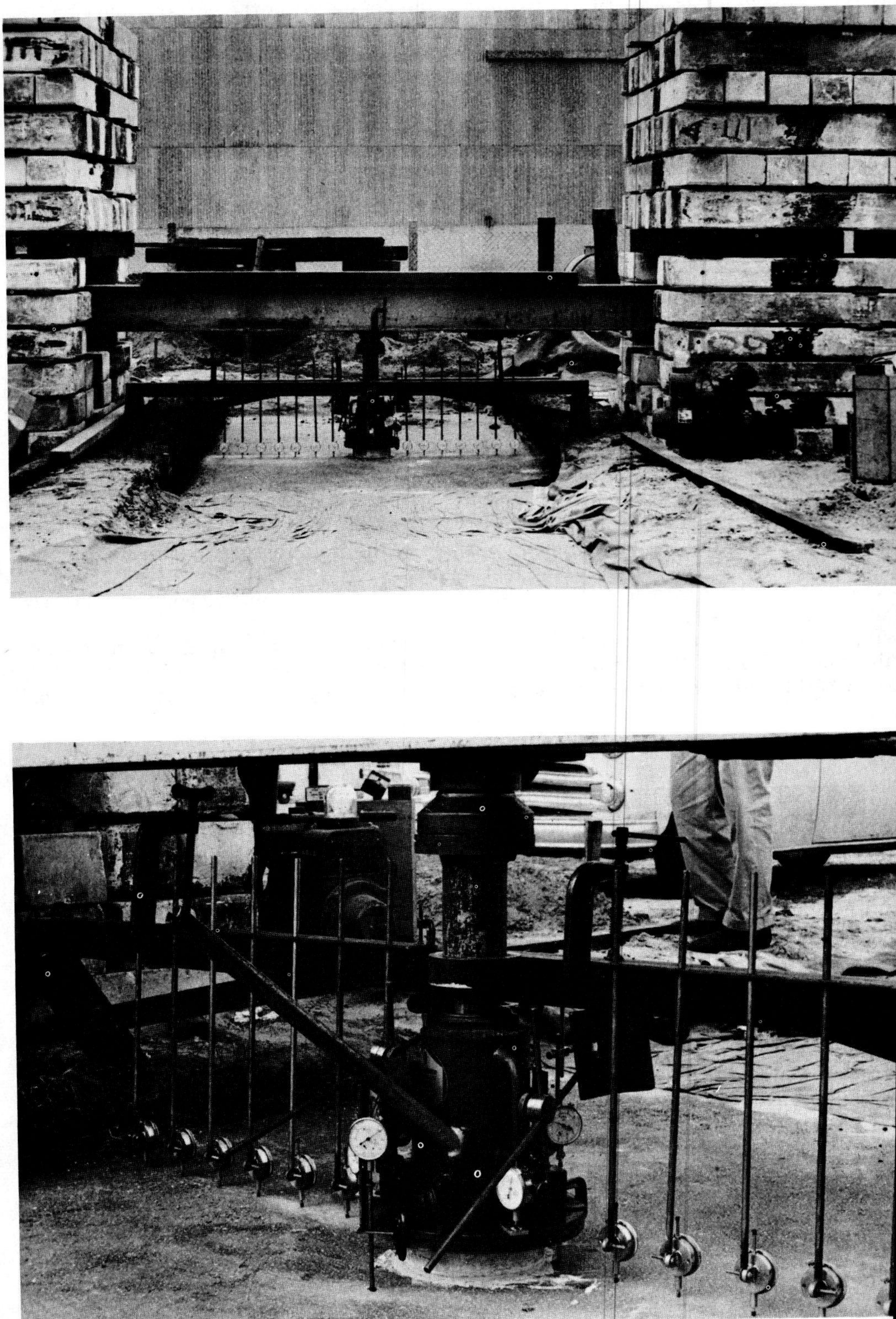


Figure 4.

tion of the modulus values and a thickness of twice the diameter is recommended.

Examination of numerous pressure deflection curves indicated that a straight line relationship did not extend much beyond a deflection of 0.05 in. and this was selected for the calculations that follow. Early work used a deflection of 0.10 in., which appears to be too high for all materials, particularly limerock. Using a deflection of 0.05 in., the modulus E_2 may be calculated for a typical subbase as follows:

$$E_2 = 1.18 \frac{pr}{w} \quad F_W \qquad (5)$$

$$= 1.18 \frac{(220)}{0.05} \quad (1) \quad = 5,200 \text{ psi}$$

For this study, pr equaled the average product of the pressure (from ASTM 1196-57 and (4)) times the radius for the 4-, 6-, 8-, 10- and 12-in. diameter plates. The depth of soil tested was in all cases equal to or greater than 4r, and E_1 = 20,000 psi. Tests are being performed during the summer of 1960 to evaluate E_1 for different base materials. Bearing tests are run in a 7-ft x 7-ft pit using a 12-in. diameter plate. Base thicknesses are increased from 4 to 24 in., the latter thickness being used to compute E_1. Using this technique, the modulus value as well as the effect of varying the thickness, h, may be investigated. Two-layer influence curves are being prepared for typical systems.)

Having evaluated E_2 and E_1 and knowing the geometry of the section to be studied, values of deflection, w, or pressure, p, for a given deflection may be computed from Burmister's equation:

$$w = 1.18 \frac{pr}{E_2} \qquad F_W \qquad (6)$$

in which F_W is obtained from Figure 3.

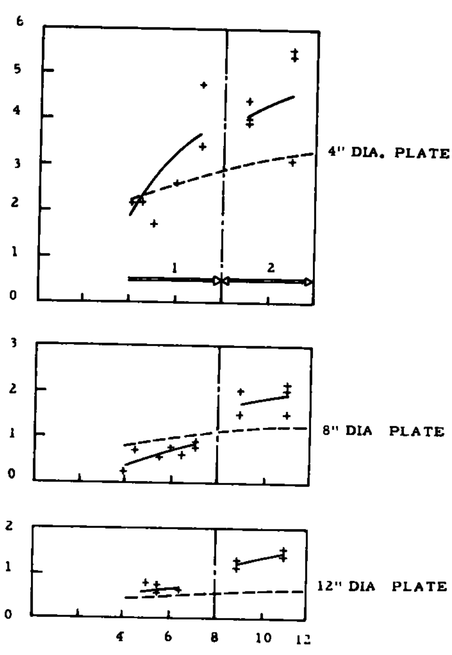


Figure 5. Experimental and theoretical curves—US 441—2-layer system study (layer I, E_1 = 20,000 psi, layer II E_2 = 5,200 psi).

As part of the two-layer study, tests were conducted in a test pit as well as on US 441. Tests in this series were performed with 4-, 8- and 12-in. diameter plates and followed a procedure similar to ASTM Standard 1196-57.

The effect of varying plate size as well as base thickness is shown in Figure 5. The curves have been developed from US 441 test data. Agreement between theory and test is fair. The 4- and 8-in. plate test curves cross the theoretical curve showing minus and plus variation. This may be attributed to the fact that when the thicker base sections were constructed in two layers (2 in Fig. 5), density may have increased which would increase E_1. Increasing E_1 from 20,000 to 25,000 psi for the test on the double lift base sections would result in reasonably good agreement between theory and test. This magnitude of increase is definitely possible.

The results of field tests conducted up to the present time indicate that the use of layered theory is quite promising. Some adjustment of the constants used in the Burmister theory may be necessary to predict the exact results obtained in the field. This is expected inasmuch as the degree with which real conditions may agree with the idealized conditions is one of the major problems associated with the use of the theoretical equation of Burmister.

THICKNESS OF WEARING SURFACE

The most recent work completed, dealt with the effect of increasing the wearing surface thickness and studying the effects on the strength and deformation characteristics of a two-layer system. The complete section was then subjected to repetitional loads. A Type-I asphaltic concrete surface was used in the research study and was tested as described in a recent report (8).

The effect of adding layers of wearing surface of 1.5-, 3-, and 4.5-in. total thickness to a limerock base 24 in. thick did generally follow the layered system concepts. The data indicate that the actual experimental deflection values are almost equal to the predicted values and the variation that exists between the different thicknesses of surfacing is within the range of experimental error. It appears that the two-layer theory is reasonable for predicting the behavior of the system investigated. Table 3 gives a comparison of experimental and theoretical data. Deflection values are given for a

TABLE 3

EXPERIMENTAL AND THEORETICAL DATA OBTAINED FROM 8-IN. DIAMETER PLATE TESTS PERFORMED ON ASPHALTIC CONCRETE SURFACES OVER LIMEROCK BASE, 1960

Condition	Deflection of Plate (in.) for Surface Thickness of 1.5 In.	3.0 In.	4.5 In.
Experimental	0.055	0.058	0.053
Theoretical[1]	0.053	0.051	0.049

[1]Surface thickness as noted;
24-in. limerock base;
8-in. diameter rigid plate; p = 200 psi;
E_2 = 17,000 psi; and
$E_2/E_1 = 1/1.6$, $\mu_1 = \mu_2 = 0.5$.

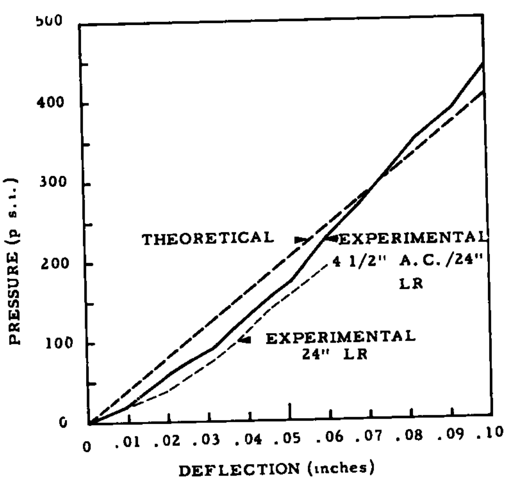


Figure 6. Pressure deflection curves for $4\frac{1}{2}$-in. asphaltic concrete, type I over limerock (LR) base, pit tests (E_2 = 17,000 psi, $E_2/E_1 = 1/1.6$, $u_1 = u_2 = 0.5$).

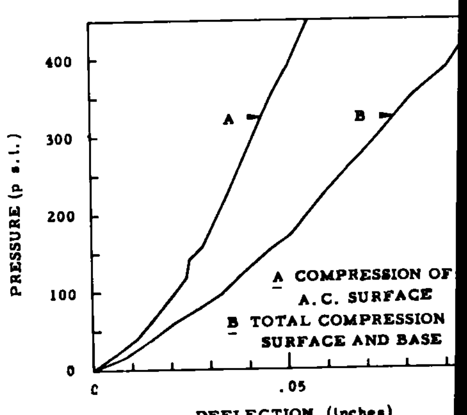


Figure 7. Compression of asphaltic concrete surface ($4\frac{1}{2}$ in.) and of limerock base (24 in.), 8-in. diameter plate (E_2/ $E_1 = 1/1.6$).

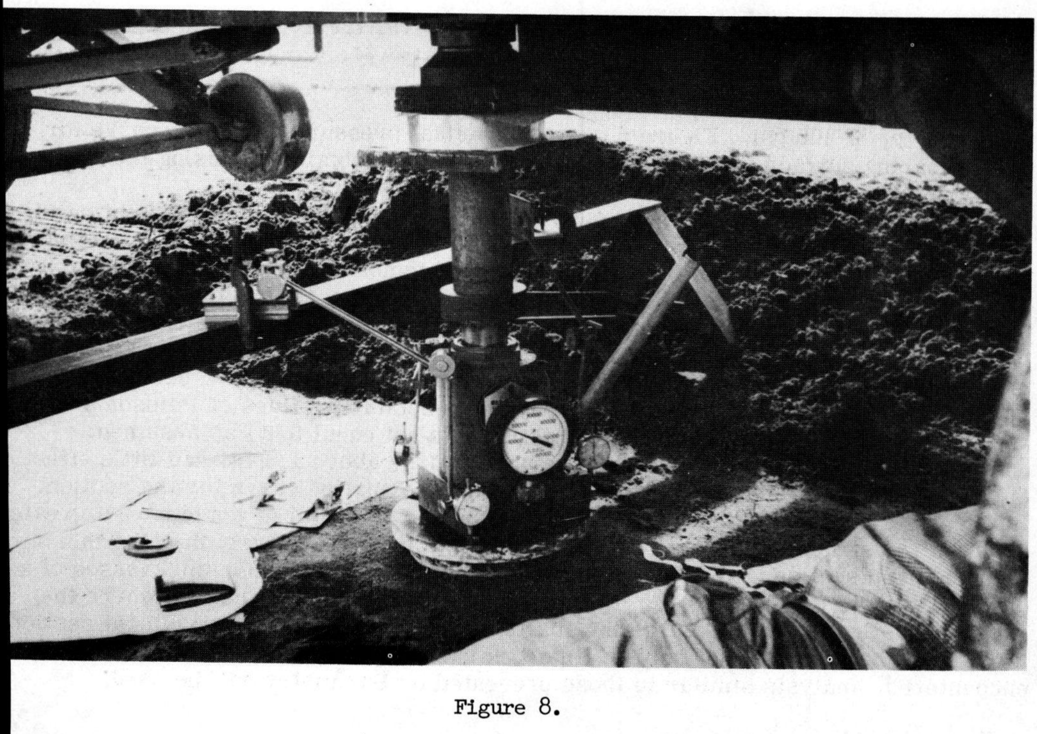


Figure 8.

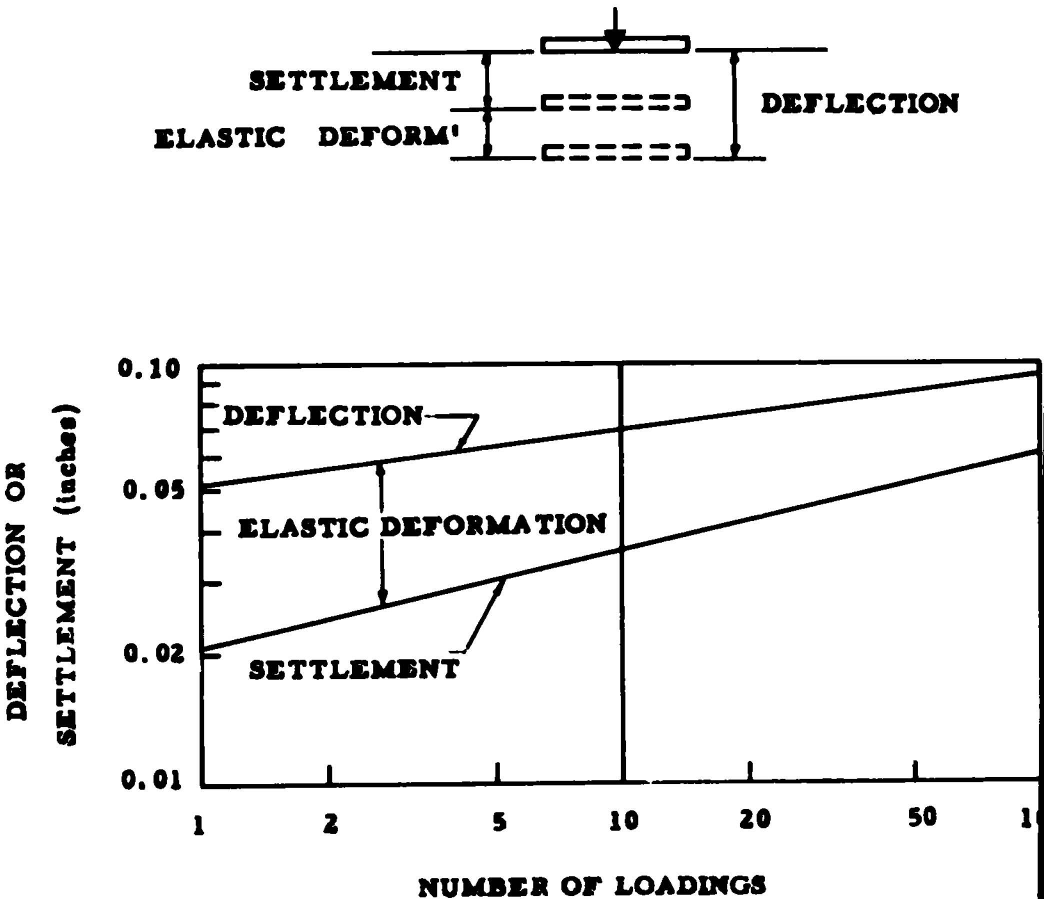


Figure 9. Increase in deflection and settlement with repetitions of a 12,000-lb load 8-in. diameter plate (8).

pressure, p, of 200 psi. Figure 6 shows the actual pressure deflection curve for 4. in. of wearing surface as well as the theoretical curve obtained by using the two-lay theory.

This study clearly indicated the need for precise measurements of deflection whe conducting this type of experiment. Accurate evaluation of the variables affecting th action of the layered system is also necessary to compare test results and theory. The theoretical computations were based on an estimate of the ratio of E_2/E_1 obtaine from modified CBR tests. This is at best only an estimate and more exact values of the modulus of asphaltic concrete are necessary before any definite conclusions can made with regard to the use of layered theory for predicting the real behavior of we ing surfaces. Experimentation is also needed to establish values of Poisson's ratio, μ, for asphaltic concrete as in all probability μ is not equal to μ_2 as assumed.

An increase in the thickness of the surface course above 1.5 in. had little effect the slope of the straight line portion of the load deformation curve for the section. However, it is probable that the thicker wearing surface course would have a greate ultimate resistance and resist the shearing stress more effectively than the thin sur face and base section. Additional tests are necessary with thinner thicknesses of as phaltic concrete (0.75 in. to 2.5 in.). Additional tests are also needed where the range of E_2/E_1 is varied to cover the limits encountered on typical pavement sectio throughout the state and not only on limerock bases. Where three-layer systems ar encountered, analysis similar to those presented by Burmister will be used.

The action of the combined section of asphaltic concrete and limerock base was that of a layered system. Figure 7 shows the deflection both of the upper surface of the asphaltic concrete and of the surface of the limerock. Proportional amounts of deflection exist throughout the deformation range tested.

Measurements were also made of the surface deflections surrounding the 8-in. diameter plate when subjected to a pressure of 80 psi. The deflected surface was typical of a layered system and extended outward from the center of the load a distance of about four diameters. The deflection curve was almost parabolic and was similar to the surface deflection curves obtained when performing Benkelman beam tests on similar pavement sections.

Figure 8 shows the field test arrangement used to obtain the compression and deflection data for this study.

REPETITIONAL LOADS

One of the major problems is that of limiting the accumulated settlements associated with repetitional loads. Extensive studies have been made and discussed by McLeod) on the effects of repetitional loads on settlements. As part of one of the bearing plate investigations a preliminary testing program was completed where 30 repetitions of load (stress = 234 psi) were applied to a pavement section consisting of 4.5 in. of asphaltic concrete over 24 in. of limerock base. The results are shown in Figure 9. The findings agreed with those obtained by McLeod. The relationships of deflection, settlement and elastic deformation are summarized in this figure for an 8-in. diameter plate.

The extrapolation of the curves beyond the 30 repetitions to 100 appears to be justified. Extrapolation beyond this range into the higher numbers of repetitions cannot be made or justified at this time. Additional tests must be made in the range of 1,000 and 10,000 repetitions to establish the settlement relationships. Plans for building repetitional load testing equipment have been made and tests should be initiated in 1961.

The repetitional load equipment will permit evaluation of accumulated settlement under repetitional loads as well as the effective soil modulus. The use of repetitional load information along with layer system analysis should lead to a more realistic method of analysis of flexible pavements.

ACKNOWLEDGMENTS

The studies discussed were sponsored by the Florida State Road Department and performed in cooperation with the Division of Research and In-Service Training. Most of the work was performed by F.C. Collins, Jr., L.J. Langfelder, and E.W. Cavin, while graduate students in the Civil Engineering Department of the University of Florida, under the author's direction.

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Condition Surveys Used in Oklahoma to Evaluate Flexible Pavement Design

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THIS PAPER outlines the procedure of making flexible pavement condition surveys nd its use in evaluating the flexible pavement design of the Oklahoma Highway Depart- ent.

In cooperation with the Bureau of Public Roads the Oklahoma Highway Department itiated a comprehensive research study in 1955 to evaluate the flexible pavement de- ign adopted in 1947. The thickness design adopted in 1947 uses the California Bear- g Ratio curves as given in HRB Proceedings, Vol. 22.

The pavement studied was selected from a list of projects consisting of 2,388 mi of exible pavement constructed after the rational method of design had been adopted in 947. Analysis of the projects indicated that five principal types of construction had een used. The mileage of each type of construction in the sample selected was in roportion to the total miles of each type of construction. The selected sample con- sted of 321 mi of two-lane pavement which had been constructed under 42 separate ntracts.

There are twelve soil problem areas in Oklahoma—so designated because the agri- ltural problems are similar throughout each area. Of these twelve areas, there are e major areas which encompass approximately 80 percent of the state. This was the cond consideration in the selection of a test sample. The 42 projects selected for udy were located within the five major problem areas. No other consideration was en to the selection of projects than those previously mentioned.

The study consisted of completing a testing program to evaluate the performance of e pavement and the compilation of historical and environmental data to be analyzed connection with the testing program. Procedures were written for assembling the ta, for analyzing it, and for making all tests. Some 40 items were included which y be summarized into five general classes.

1. Construction Data.—The items in this group included information taken from the nstruction plans; such as, typical pavement section, type of construction, and qual- tests of materials made during construction.
2. Other Existing Data.—This group included geology, weather, original soil sur- ys, traffic data, and maintenance costs.
3. Field Data.—This group included condition surveys of the pavement structure, ghometer surveys, field checks of the original soil surveys, and pedological soil veys.
4. Field Tests.—Included plate bearing tests, Benkelman beam deflection tests, d California Bearing Ratio tests, density tests, moisture tests, and the taking of nples for laboratory testing.
5. Laboratory Tests.—Included routine laboratory testing to determine whether d samples conformed to specifications for the subbase, base, and surface courses.

Many factors determine the performance of the pavement structure. It was intended nclude in this study all the principal factors that could possibly be evaluated. It was med necessary to obtain a factor for evaluation purposes which could represent the reciation of the pavement structure. Expended maintenance funds for the pavement ucture were considered as partial payment for depreciation. The present condition he pavement structure was considered as the other part of depreciation.

To begin the study of depreciation, maintenance costs of each of the 42 construction projects, as indicated by statistical records, were tabulated and reduced to 1950 costs. All maintenance costs were then converted to a factor which represented the average cost per mile per year for each project. The average cost per mile for the contract construction of the pavement structure was obtained and converted to the 1950 cost. The average maintenance cost per mile per year was divided by the average cost per mile for contract construction, 1950 cost, to obtain a percentage factor which represented repaired depreciation. As previously mentioned, the present condition of the pavement structure was considered as the other part of depreciation. Unrepaired depreciation can be estimated by a condition survey and can be expressed as a percentag of the cost of the pavement structure. Condition surveys require an estimation founde on the judgment of the individual, and the personal factor is a major consideration.

For an observer to pass over an extent of pavement and mentally total up and reduc to an exact figure a number of areas of several kinds of defects, is an ability that will differ greatly among individuals. For long extents and many items, this ability probably varies greatly in the same individual at different times. To minimize the person al factor it is advisable to divide a project into a number of small parts and to evaluat each part separately. The final condition rating of the project can be made by averag- ing the evaluation of the parts.

To begin the condition surveys, reference points were painted on the surface of the pavement at each 0.2-mi longitudinal interval throughout the length of the project and numbered in consecutive order from the beginning. The exact stations from the construction plans were determined for each of the reference points. The reference poi were used as ties for the condition survey, soil and geological surveys, Benkelman beam deflection sites, and plate bearing sites.

To minimize and standardize the personal factor for rating purposes in making the condition survey, the following terms, classifications, and ratings were adopted. De inition of the terms used in describing the different characteristics of the classes is as follows:

Terms	Percent of Area
Few - slight	Less than 5
Some	5 to 15
Considerable	15 to 30
Extensive	More than 30

The percentages are given as part of the total area of the extent rated. The class ratings, and definition of the characteristics of the classes are as follows:

Excellent (98-100 percent)

1. No major or minor defects are apparent.
2. No maintenance has been performed.

Superior (90-97 percent)

1. There are no base failures or other major defects.
2. No structural maintenance has yet been necessary.
3. Any one or all of the following characteristics may be present within a 0.2- mi extent: (a) slight surface roughness; (b) slight cracking; and (c) the riding quali is impaired but very slightly.

Good (80-89 percent)

1. No base failures.
2. Any one or all of the following characteristics may be present within a 0.2 mi extent: (a) some surface roughness; (b) some cracking; (c) slight raveling; and (d) slight distortion.

Any one or all of the characteristics listed in the following classes may be prese within a 0.2-mi extent:

Average (65-79 percent)

1. Few localized base failures.

2. Considerable surface roughness.
3. Considerable cracking.
4. Some raveling, especially in the outer wheel lanes and along the edges.
5. Some distortion.

Poor (50-64 percent)

1. Considerable base failures.
2. Extensive surface roughness.
3. Cracking is extensive.
4. The surface has raveled extensively throughout its width.
5. Considerable distortion.

Failure (Less than 50 percent)

1. Base failures are numerous and extensive.
2. Distortion is extensive.
3. Traffic hazards are extensive due to failures and distortion.
4. Routine and special maintenance repairs have not been effective.

If maintenance had been performed, the maintained area was rated in one of the pre-
eding classifications as to its effectiveness. A note was made in the remarks column f the condition survey form regarding the type of maintenance that had been performed. ther remarks included the general condition of the pavement structure. The final con-ition rating of a project was obtained by averaging the ratings of each 0.2 miles. Fig-re 1 shows the condition survey form.

A glossary of terms used in the condition survey follows:

Pavement Structure: The traveled portion of the road consisting of the subbase, base, and surface.

Surface Roughness: Inequalities in the pavement surface which adversely affect the riding quality.

Cracks: Approximately vertical cleavage due to natural causes or traffic action.

A. Transverse cracks—a crack which follows an approximate course at right angles to the centerline.

B. Longitudinal cracks—a crack which follows an approximate course parallel to the centerline.

CONDITION SURVEY DATA

ta taken by ____________ Date ________ Control Section ____________

oject # ____________ Research Group # ______ Research Project # ______

unty ____________ H'way US SH ______ Length ____________ Miles

oject Description & Location ______________________________

______________________________ Date Started ________

Date Completed ________

hicle # __________ Mileage Conversion Factor __________ Final Rating ______

eter ading	Acc'l. Mileage	Corrected Mileage	Defl. No.	Defl. Type	Cond. Rating	Remarks

Figure 1.

C. Shrinkage cracks—interconnected cracks forming a series of large polygons usually with sharp corners or angles.

D. Slippage cracks—frequently crescent-shaped cracks which usually point in the direction of the thrust of traffic.

Stripping: The separation of bituminous films from aggregate particles.

Raveling: The progressive disintegration of the surface by the dislodgement of aggregate particles.

Distortion: Any type of irregularity tending to distort the pavement surface from its original shape.

A. Corrugations—transverse undulations at regular intervals in the surface of the pavement consisting of alternate valleys and crests not more than 2 ft apart.

B. Waves—transverse undulations at regular intervals in the surface of the pavement consisting of alternate valleys and crests 2 ft or more apart.

C. Rutting—the formation of longitudinal depressions under traffic in the wheel lanes.

Failure: Disintegration of the pavement structure.

A. Alligator cracking—interlaced cracking of a bituminous surface course into small irregular blocks caused by inadequate base support.

B. Shoving—lateral displacement of the pavement material due to the action of traffic.

C. Disintegration—deterioration into small fragments or particles due to any cause.

D. Potholes—bowl-shaped holes of varying sizes in the pavement resulting fro localized disintegration.

After the completion of the condition survey, the average condition rating of the project was computed and divided by the age of the project to obtain the average condi tion depreciation per year. This factor was considered as the unrepaired depreciatio percentage and added to the repaired depreciation factor to obtain the total depreciati per mile per year as a percent of the contract construction cost based on 1950 costs.

The depreciation per mile per year of the pavement structure was used as a basic factor in the study to determine the relationship and effect of the following:

1. Load supporting ability of the pavement structure as determined by plate bearing tests and Benkelman beam deflection tests.
2. Thickness of the "as built" pavement structure.
3. Traffic and wheel load densities.
4. Soil and geological extents.
5. Climatic conditions.
6. Quality of subbase, base, and surface courses of the pavement structure.
7. The original construction cost of the pavement structure.
8. The maintenance cost since completion of the pavement structure.

Although this study was started in 1955, the complete analysis has not as yet bee completed. The relationship of items 2, 3, 4, 5, 7, and 8 to depreciation has been determined and is included in Part One of the Final Report of the Oklahoma Flexible Paving Research Project, 1958. Analysis is under way as to the effect of each of th items to total depreciation and will be published in Part Two of the Final Report whe completed.

The procedure described herein for making condition surveys was found to give reasonably good results. It was developed in 1955 prior to the first condition surve of the 42 projects. The procedure has been used for making surveys of the same pr jects in 1957, 1959, and 1960.

The average condition depreciation per mile per year of the 42 research projects is as follows:

Date of Survey	Average Age of Projects, Yr	Condition Depreciation, %	Average Condition Rating
une 1955	4.402	1.43	91.92
une 1957	6.285	2.05	87.12
une 1959	8.154	2.00	83.73
une 1960	9.154	2.47	77.30

Since the original condition survey was made in 1955, maintenance consisting of ingle bituminous surface treatments has been placed on 19 of the 42 projects. The re- ılts of the condition surveys indicate that the pavements are depreciating at a more ıpid rate than was anticipated and maintenance performed has not been adequate.

The rapid depreciation also indicated that the pavement structure was underdesigned r the poorest soil types and resulted in the development of an interim design method, lopted in 1958, which extended the design curve to give greater thickness of the pave- ent structure for the poorest soils. The interim design method consisted of the de- lopment of a subgrade index number ranging from 0 to 40 for soil characteristics pendent on the plasticity index, liquid limit, and percent passing the No. 200 sieve. ıe relationship between the subgrade index numbers and the California Bearing Ra- ɔs of the soils was determined, and the appropriate pavement thickness was deter- ined from standard CBR curves for subgrade index numbers. The subgrade index mber was then used in place of the standard CBR curves.

Preliminary analysis indicates that factors other than strength of the subgrade soils ect the performance of the pavement structure and inadequate design results from lure to provide for the other factors. Climatic environment, traffic, and wheel load nsity are among the chief factors affecting the performance. One project included the study gave almost perfect performance for approximately eight years while pre- itation was below normal and then depreciated 38 percent in three years when rain- l exceeded the normal average.

Another project on a secondary road gave good performance until heavy truck loads asphalt were moved over it.

Another project performed good for a period of time and then the edges started fail- , probably due to a lack of shoulder width.

The condition survey, which resulted in the calculated depreciation, is being used a basis for evaluation of flexible pavements. The relationship of depreciation to the ny factors affecting performance is being determined by machine analysis. The end ult of the study will be a mathematical regression equation, including major factors, designing flexible pavement thickness.

Non-Dimensional Techniques Applied to Rigid Plate Bearing Tests on Flexible Pavements

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Non-dimensional techniques based on the methods of dimensional analysis provide a rational basis for analyzing rigid plate bearing tests on flexible pavements. Test data reported by Benkelman and Williams (1, Table 4) have been successfully analyzed by such techniques. The surface deflection is explicitly expressed as a function of the applied load, bearing plate diameter, pavement thickness, and the strength characteristics of the subgrade. Several illustrative examples are presented using the derived deflection equation to indicate possible applications. For the Hybla Valley data analyzed, the analysis shows that the load-carrying capacity of the flexible pavement as expressed by the surface deflection is dependent on the total pavement thickness and not on its proportion of asphaltic concrete or base course.

● ONE of the most comprehensive field investigations of rigid plates bearing on flexible pavements is a cooperative study conducted by the U.S. Bureau of Public Roads, the Asphalt Institute and the Highway Research Board on a specially constructed track at Hybla Valley near Alexandria, Virginia. The factual test data of the study are presented in tabular form by Benkelman and Williams (1).

Included are static rigid-plate bearing tests on full-scale pavement sections constructed on a minimum embankment of 5 feet of uniform A-7-6 soil (AASHO Classification—1949). The test sections of pavement were built with great care and every precaution was exercised to insure uniformity of thickness, compaction and composition of the various component layers. The soil used in the embankment was secured from a previously prospected area and a high degree of uniformity of the material, both in composition and condition, was obtained. The first stages were completed in 1946, whereas some sections were not placed until 1949.

There are innumerable possible procedures for conducting static load tests. For any given pavement section the various controllable factors that may affect the results of tests of this type include the magnitude of the load and the manner in which it is applied, the number of applications and releases of a given load, the duration of each load application and release, and the size of the bearing plate. The data presented were obtained by the use of four different load-test procedures; namely, the incremental, the incremental-repetitional, the accelerated and the repetitional. The vast majority of the tests were made with the accelerated and repetitional procedures.

The incremental tests were conducted on 3-, 6-, and 9-in. asphaltic concrete surface courses of a 24-in. base section using circular bearing plates of 1.954- and 3.568-in. diameters. The relatively small plate diameters, compared to the thickness of the surface course, confined the effects of the applied load to the surface course. Therefore such tests do not give a true indication of the load-carrying capacity of the pavement section. The most desirable procedure would have been to use the incremental test with 12-, 18-, 24- and 30-in. diameter bearing plates for surface deformations up to approximately 1 or 1.5 in.

The accelerated test procedure was conceived in a search for a method that would produce the data sought and at the same time permit the conduct of a number of tests

oer day. It consists of two parts, an incremental portion (part a) which is a much ab-
oreviated version of the actual incremental test procedure mentioned, and an acceler-
ated portion (part b). The incremental portion provides for the application and release, once each, of three individual loads of increasing magnitude. The period of application or release is maintained until the rate of movement slows to 0.001 in. per 15 sec.. The oad magnitudes are such as to produce gross deflections of approximately 0.20, 0.30 nd 0.40 in. for each of the three loads, respectively. Following the release of the hird load and after the movement-time criterion of the incremental portion has been atisfied, the accelerated portion of the test is begun. It consists of the continuous ap-lication of a load of varying magnitude which is controlled so as to produce a rate of ertical movement of the surface under test of 0.5 in. per min. The application of the oad is continued until (a) the material is unable to support a further increase, or (b) ne gross deflection exceeds 2.0 in. or (c) the total reaction load is used.

TABLE 1

FLEXIBLE PAVEMENT SECTIONS AND PLATE SIZES ANALYZED

hickness of Asphaltic Concrete a (in.)	Thickness of Base Course b (in.)	Bearing Plate Diameter d (in.)			
		12	18	24	30
3	6	x	x	x	x
3	12	x	x	x	x
3	18	x	x	x	x
3	24	x	x	x	x
6	6	x	x	x	-
6	12	x	x	x	-
6	18	-	x	x	x
6	24	-	x	x	x
9	6	x	x	x	-
9	12	x	x	x	-
9	18	-	x	x	x
9	24	-	x	x	x

Because the accelerated procedure is actually two types of test—an incremental, d creep test (quasi static) followed by a constant rate of deformation test—it is not isable to use the results for deformation greater than 0.4 in. which is the limiting ormation for the incremental portion. It would be possible to analyze a constant e of deformation-type test if the incremental had not been conducted first.

The incremental-repetitional tests were conducted on subgrades and the repetitional s covered only small deformations. In addition, the tests conducted on the base course n the asphaltic concrete removed were influenced by the confining effect of the surface rse.

After carefully examining the various test procedures, it was decided that the incremen-part of the accelerated tests given in Table 4 of HRB Special Report 46 (1) for the complete ement section is the most meaningful data and as such is the only data analyzed in this paper.

One method of analysis using the results of the constant rate of deflection portion of accelerated procedure was presented by Ingimarrson (3), in which the linear e-ion of Housel's perimeter-shear theory is shown to be applicable. Ingimarsson's ification of the constant rate of deflection data to eliminate the effects of the pre-ng incremental portion is questionable. The results of Ingimarsson's paper are sented in terms of Housel's "perimeter-shear constant" and "developed-pressure stant" which are plotted as functions of the surface deflection. However, these re-

TABLE 2

PHYSICAL QUANTITIES CONSIDERED FOR THE RIGID BEARING PLATE TESTS ON FLEXIBLE PAVEMENTS

Physical Quantities	Symbol	Fundamental Units
Surface deformation	x	L
Total applied force	F	F
Thickness of asphaltic concrete	a	L
Thickness of asphaltic concrete plus subbase	h	L
Cross-sectional area of the bearing plate	A	L^2
Perimeter of the bearing plate	c	L
Time	t	T
Maximum unconfined compressive strength of the soil	τ	FL^{-2}
Viscosity of the soil	η	$FL^{-2}T$
Characteristic strength parameter of the asphaltic concrete	k_1	FL^{-2}
Characteristic viscosity of the asphaltic concrete	c_1	$FL^{-2}T$
Characteristic strength parameter of the subbase	k_2	FL^{-2}
Characteristic viscosity of the subbase	c_2	$FL^{-2}T$

sults are not expressed explicitly in terms of the parameters pertinent to the study.

It is the purpose of this paper to study this same data by non-dimensional techniq based on the variables involved in the investigation, and to develop an explicit functi al relationship among these variables.

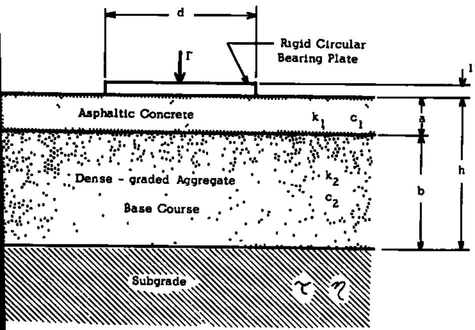
THEORETICAL DEVELOPMENT

Although some of the concepts of dimensional analysis go back to the time of Gali and have been used in various ways by such investigators as Mariotte, Newton, Fou Stokes, Froude, Reynolds, Rayleigh, and others (3), the basic theorem was not forr ly presented and proved until 1914 by Buckingham (4) in his famous Pi Theorem. A general proof has more recently been given by Martinot-Lagarde (5). The general th of dimensional analysis has been illustrated by numerous authors, particularly in th field of fluid mechanics, and several books have been written on the subject; for ex ple, Bridgman (6), Murphy (7) and Langhaar (8). At present the senior author has b applying such techniques to a variety of problems in the field of soil mechanics (9, 1 11, 12, 13, 14, 15). Because of the complex properties of the various pavement ma rials and the complicated interaction of these various layers with the loads being su ported, it is felt that the use of non-dimensional techniques in both model and protot research investigations of pavement problems would offer definite advantages with r gard to the cost, scope, and time for completion of such studies.

Thus the study reported in this paper not only provides another analysis of a por of the Hybla Valley test data, but of more importance, it illustrates and calls attent to the possible advantageous use of such a well-known general research tool as dim sional analysis in the field of pavement design. The authors are certainly not propo any new theoretical methods, but are only calling attention to an existing research t and illustrating one way in which such techniques can be extended into the practical pects of pavement design.

Examples of the practical use of non-dimensional techniques, based on the meth of dimensional analysis, in the area of soil mechanics have been given by Kondner (10, 12, 14, 15), Kondner and Edwards (11) and Kondner and Krizek (13).

The methods of dimensional analysis as used to determine relationships between hysical quantities may be briefly summarized as follows: there are m physical quantities, containing n fundamental units, which can be related by an equation, then there are (m-n) and only (m-n) independent, non-dimensional parameters, called π terms, which are arguments of an indeterminate, homogeneous function F.



gure 1. Cross-section of flexible pavement.

$$F(\pi_1, \pi_2, \pi_3 \ldots\ldots \pi_{m-n}) = 0 \qquad (1)$$

The physical quantities given in Table 2 have been selected for use in the dimensional analysis of the problem of the rigid plate bearing test on the surface of a flexible pavement. A force, length, time system of fundamental units has been used. Figure 1 is a typical cross-section of a flexible pavement showing the bearing plate, asphaltic concrete layer, base course and subgrade.

It is assumed that the material constants needed to describe the deformation characteristics of the cohesive soil subgrade are implicit in a characteristic soil strength rameter and the viscosity. The characteristic soil strength parameter used is the ximum unconfined compression strength of the soil. It may very well be that for the nge of surface deflections being considered in this paper that the problem is primar- one of deformation and not of failure. As such the soil moduli in compression and ear should be used instead of the shearing strength as given by the unconfined com- essive strength, but with regard to practical application these quantities are not as sily obtainable as the unconfined compressive strength. In addition previous work by senior author (10) on stress relaxation and creep characteristics of a cohesive soil licate that compression and shear moduli tend to be proportional to the maximum un- nfined compressive strength. The viscosity controls the rate at which the deforma- n takes place and may include non-Newtonian effects. It is also assumed that the de- mation characteristics of the asphaltic concrete and the subbase are each controlled characteristic strength parameters and viscosities. The duration of loading is im- rtant in creep and viscous response. The effect of the geometry of the bearing plate expressed by the cross-sectional area and the circumference.

Because there are thirteen physical quantities and three fundamental units, there st be ten independent, non-dimensional π terms. By a methodical process pre- usly described by Kondner (9, 10, 11, 12) the following π terms can be obtained:

$$\pi_1 = \frac{F}{A\tau}, \quad \pi_2 = \frac{c^2}{A}, \quad \pi_3 = \frac{x}{c}, \quad \pi_4 = \frac{c}{h}, \quad \pi_5 = \frac{a}{c}, \quad \pi_6 = \frac{\tau t}{\eta}, \quad \pi_7 = \frac{k_1}{\tau}, \quad \pi_8 = \frac{k_2}{\tau},$$
$$\pi_9 = \frac{k_1 t}{c_1}, \quad \pi_{10} = \frac{k_2 t}{c_2} \qquad (2)$$

The above π terms can be substituted into Eq. 1 to obtain the function F.

A general interpretation of these non-dimensional parameters has previously been en by Kondner (10, 12). The terms π_1, π_7, and π_8 express the strength ratios of the grade, asphaltic concrete, and base course, respectively. The ratios of the time of ding to the relaxation time for the subgrade, asphaltic concrete, and base course are en by π_6, π_9, and π_{10}, respectively. The term $\frac{c^2}{A}$ is a shape factor, and $\frac{c}{h}$ and $\frac{a}{c}$ characteristic length ratios. For circular- and square-shaped plates the value of s 4π and 16, respectively, regardless of the size. The settlement parameter is en as $\frac{x}{c}$ and is the dependent parameter for the study.

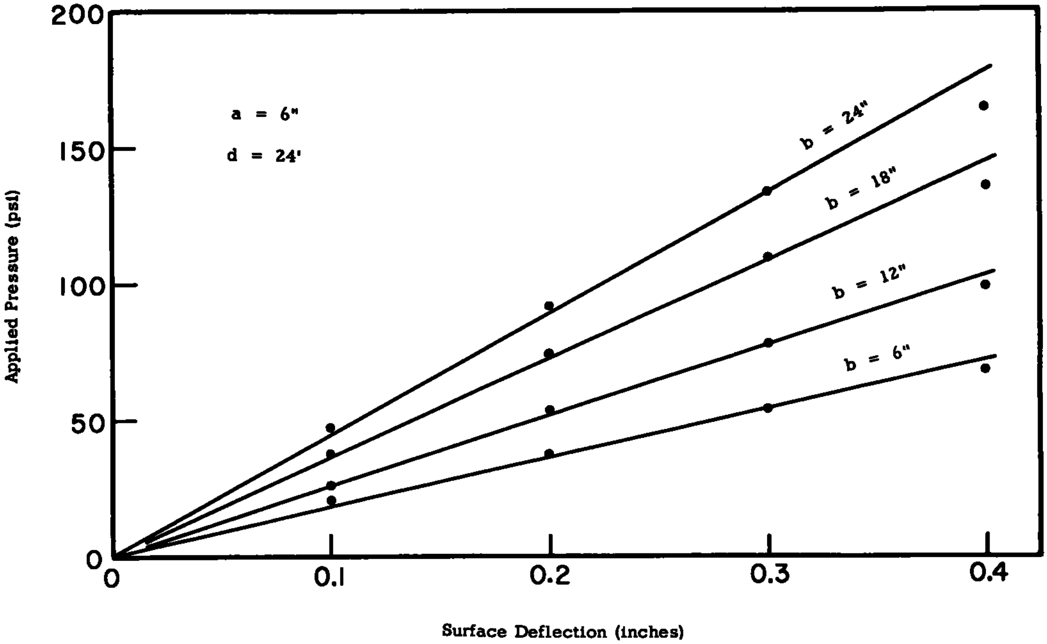


Figure 2. Applied pressure versus surface deflection.

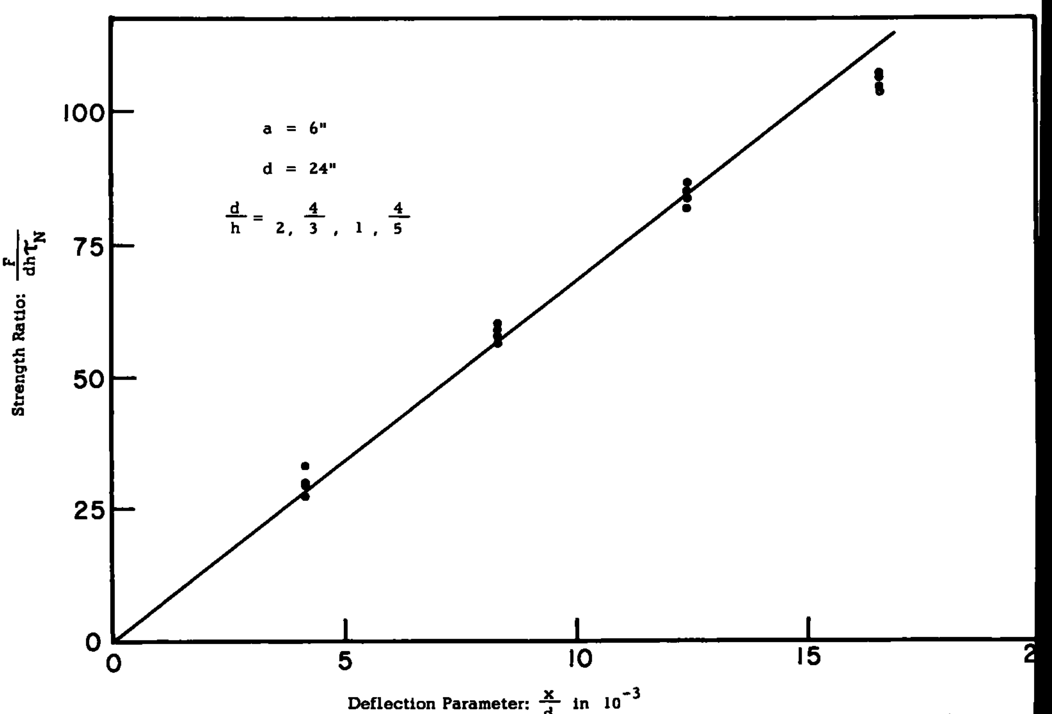


Figure 3. Non-dimensional plot: strength ratio versus deflection parameter.

The functional relationship given by Eq. 1 can be written as:

$$\frac{x}{c} = \theta \left[\frac{F}{A\tau}, \frac{c^2}{A}, \frac{c}{h}, \frac{a}{c}, \frac{\tau t}{\eta}, \frac{k_1}{\tau}, \frac{k_2}{\tau}, \frac{k_1 t}{c_1}, \frac{k_2 t}{c_2} \right] \quad (3)$$

EXPERIMENTAL RESULTS

For all of the Hybla Valley tests reported (1), τ, η, k_1, c_1, k_2 and c_2 were main-
ined constant and hence π_7, π_8, π_9, and π_{10} were also constant for the investigation
nd can be eliminated from Eq. 3. This does not mean that the load-deflection relation
independent of the type and quality of pavement materials, but only that the pavement
aterials were constant for the data analyzed. It is to be expected that the curves giv-
n would in general be different for different pavement materials and perhaps even for
fferent types of loading. Thus the analysis that follows is for the particular values of
, π_8, π_9, and π_{10} used at Hybla Valley. The tests were conducted in such a manner
as to minimize time effects and hence π_6 is relatively constant and can be dropped.
ecause only circular bearing plates were used in the study, the diameter, d, expresses
e geometry of the bearing plate and replaces the perimeter and cross-sectional area.
his leaves one dependent and three independent variables which can be algebraically
ansformed into the form given by Eq. 4. It is important to note that the new π terms
e the variables under consideration and not the individual physical quantities compos-
g the π terms.

$$\frac{x}{d} = \theta' \left[\frac{F}{dh\tau}, \frac{d}{h}, \frac{a}{d} \right] \quad (4)$$

Figure 2 is a typical conventional plot of applied pressure versus surface deflection
various thicknesses of base course with a bearing plate of constant diameter and a con-
nt thickness of asphaltic concrete. The four different straight lines for various val-
s of b indicate the apparent effect of the base course thickness; however inasmuch as a
a constant, the variation in b is reflected as a variation in the total pavement thick-
ss h and, because d is a constant, the variation of b is expressed as a variation of
ratio $\frac{d}{h}$. The same test results are plotted in Figure 3 in the non-dimensional form
$\frac{F}{dh\tau_N}$ versus $\frac{x}{d}$. Comparison of Figures 2 and 3 clearly illustrates the advantages
dimensional analysis as an experimental guide and the advantages of expressing ex-
imental data in non-dimensional form. Because τ was constant for all tests, the
ameter $\frac{F}{dh\tau}$ is proportional to $\frac{F}{dh}$ and is given for τ_N, a normalized value of τ e-
l to unity. Figure 3 is not affected by the variation of $\frac{d}{h}$ and hence $\frac{d}{h}$ can be elimin-
d from Eq. 4. For these data the ratio $\frac{a}{d}$ was a constant value of 0.25.

Another conventional method of presenting the data is shown in Figure 4 where the
lied pressure is plotted against the surface deflection for various values of the
kness of the asphaltic concrete with constant values for the plate diameter and the
base thickness. Note the apparent influence of the thickness of asphaltic concrete.
ause the diameter of the plate is constant, this variation can be expressed in terms
. Figure 5 is the same data plotted as $\frac{F}{dh\tau_N}$ versus $\frac{x}{d}$. The three curves of Fig-
4 are reduced to one curve in Figure 5. The same linear relationship of Figure 5
btained for base courses of 12, 18 and 24 in. with a constant plate diameter of 18 in.
repeating this analysis for plate diameters of 12, 24 and 30 in., a single resultant
ve can be obtained for each plate diameter (Fig. 6). Thus, the non-dimensional
ameter $\frac{a}{d}$ exerts very little influence on the phenomena and can be dropped from Eq.
The results of Figure 6 can also be obtained by plotting $\frac{F}{dh\tau_N}$ versus $\frac{x}{d}$, as shown
Figure 3 for d = 24 in., for all the plate diameters.

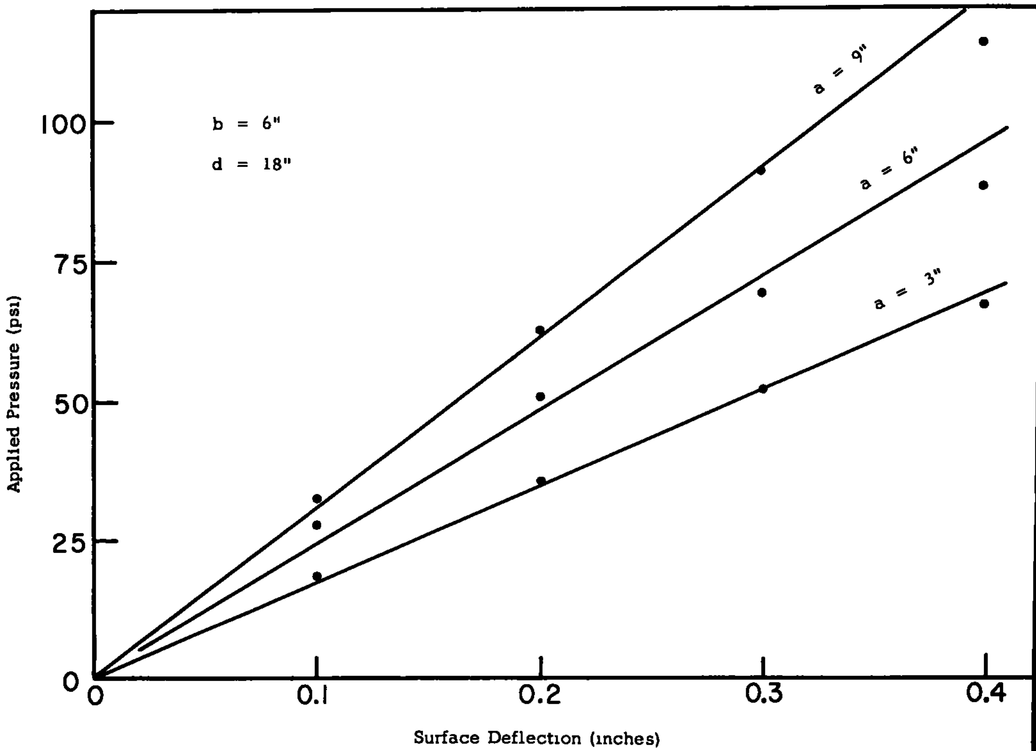


Figure 4. Applied pressure versus surface deflection.

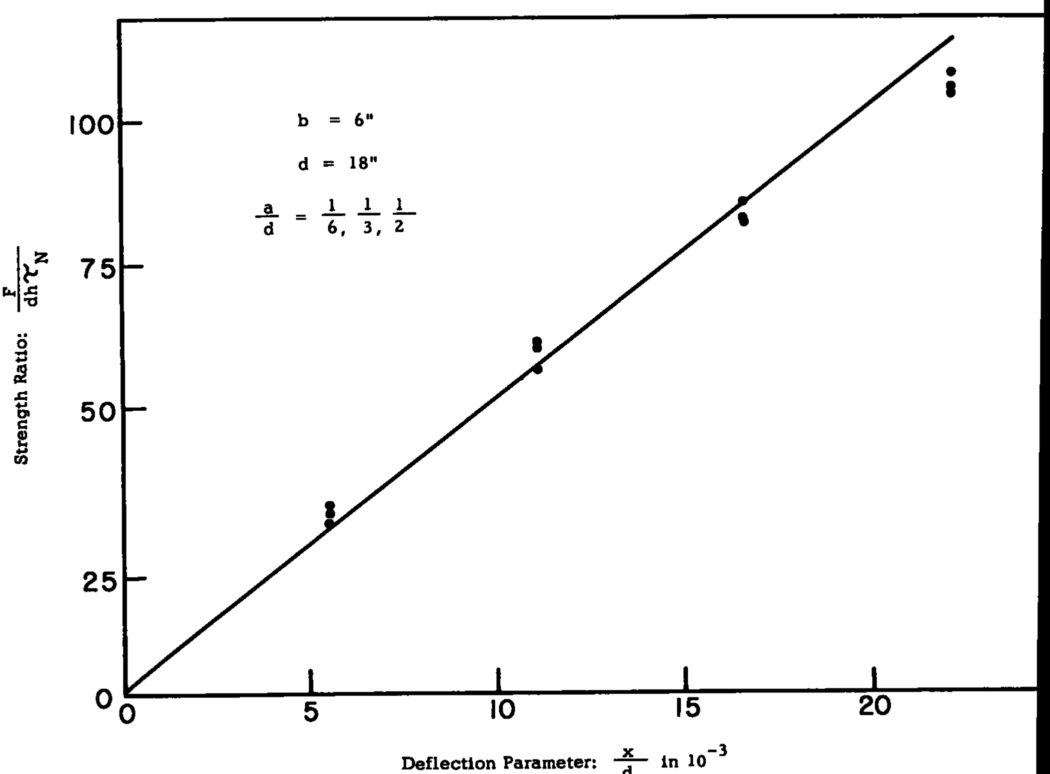


Figure 5. Non-dimensional plot: strength ratio versus deflection parameter.

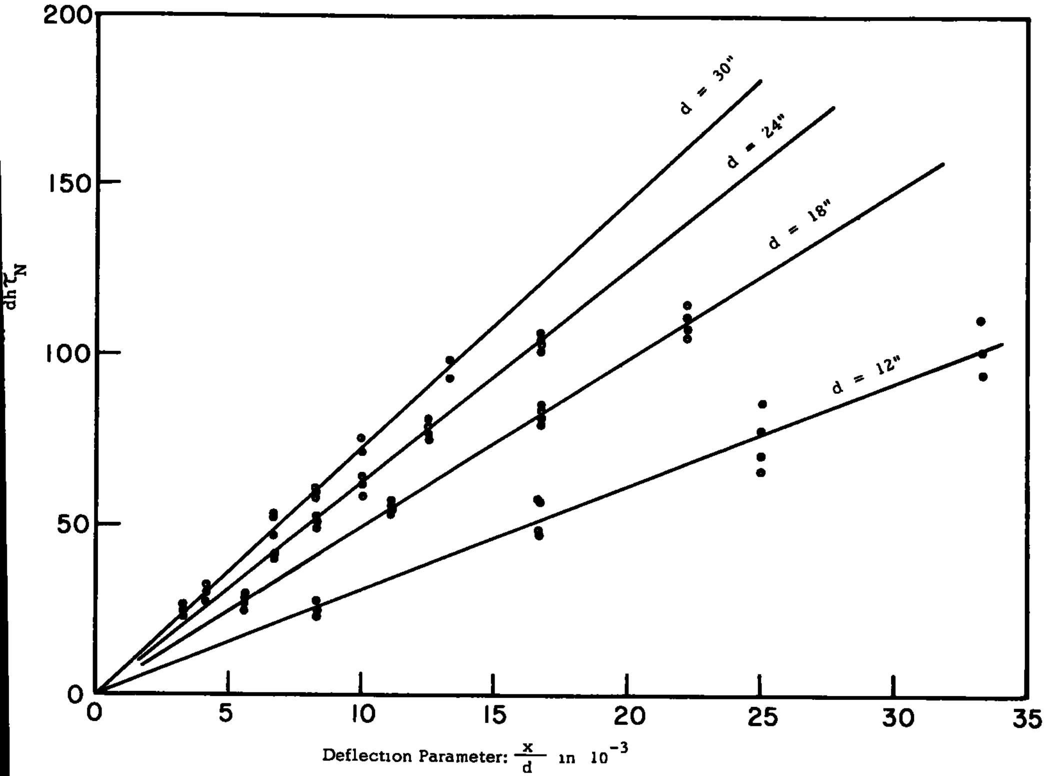


Figure 6. Strength ratio versus deflection parameter: variable plate parameter.

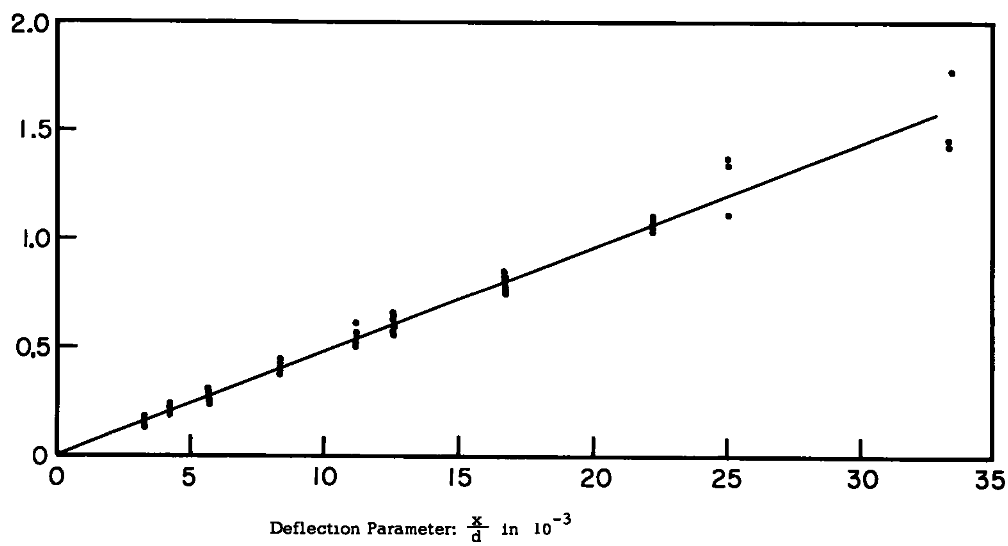


Figure 7. Strength ratio versus deflection parameter: normalized diameter.

The curves of Figure 6 can be reduced to the straight line of Figure 7 by normalizing with respect to the bearing plate diameter. Thus, the data of Table 4 (1) can be reduced to the relation given in Figure 7. Because the curve of Figure 7 is a straight line from the origin, the surface deflection as a function of the applied load, bearing plate size, subgrade strength characteristics, and pavement characteristics can be written as

$$x = M \frac{F}{dh\tau} = \frac{F}{4dh\tau} \quad (5)$$

where M includes the effect of normalizing with respect to the diameter as well as the slope of the straight line. For the Hybla Valley study considered in this paper the factor M in Eq. 5 was found to be 1/4. Because of the normalization process the factor 1/4 in Eq. 5 has the units of inches. Eq. 5 is also based on an estimated value of the maximum unconfined compressive strength of the subgrade obtained from another report on the Hybla Valley Study (16). Because $\frac{F}{dh\tau}$ is non-dimensional, any system of compatible units may be used in Eq. 5 and the value of the deflection will be given in inches.

Because of possible variations in the properties of the subgrade, asphaltic concrete and base course, the results expressed by Eq. 5 or by Figure 7 for the Hybla Valley Study may not apply to pavement sections in all localities of the country. It is felt that the basic method of analysis given in Eq. 3 and applied to the Hybla Valley data could also be applied in other localities to determine the necessary relationship to replace Eq. 5.

If the pavement response is linear as indicated in Figures 2 and 4, and if the present techniques are applicable in other localities under various conditions, the procedure required to determine the factor M would be as follows. Determine the maximum unconfined compressive strength of the subgrade and then conduct several rigid plate bearing tests using several plates of different diameter, each with several applied loads. For each diameter plate used, plot $\frac{F}{dh\tau}$ versus $\frac{x}{d}$. To reduce these plots into a single relationship, select a convenient diameter as a normalizing factor and apply it to each plot. If the single resultant plot is a straight line, its slope can be determined and divided into the plate diameter which was used as the normalizing factor in order to obtain the factor M to replace $\frac{1}{4}$ in Eq. 5. If the resultant plot is not a straight line and its equation cannot be determined, the resultant plot itself should be used.

For the case of large surface deflections involving non-linearities Eq. 3 could also be used, but the procedure involved in determining the explicit form of Eq. 3 might be considerably different.

ILLUSTRATIVE EXAMPLES

The following examples are given to illustrate the possible use of Eq. 5.

Example 1

Predict the surface deflection of a flexible pavement consisting of a 6-in. asphalt concrete surface course and a 12-in. dense-graded aggregate base course supported by a subgrade with a maximum unconfined compressive strength of 64 psi when tested in rigid plate bearing under an applied pressure, p, of 78 psi and a bearing plate diameter of 24 in.

Solution:

The total applied load is 35,400 lb and Eq. 5 gives

$$x = \frac{F}{4dh\tau} = \frac{35,400}{4(24)(18)(64)} = 0.32 \text{ in.}$$

This problem was randomly selected from Table 4 of Special Report 46 (1) and had a field deflection of 0.3 in. The predicted deflection value given by Eq. 5 is 6.7 percent higher than the recorded value.

Example 2

Determine the applied pressure, p, necessary to cause a surface deflection of 0.2 in. or a flexible pavement section of a 3-in. asphaltic concrete and a 24-in. base course ayer on a subgrade with a maximum unconfined compressive strength of 64 psi when ested with an 18 in. diameter, rigid bearing plate.

Solution:

$$p = \frac{4F}{\pi d^2} = \frac{4(4dh\tau x)}{\pi d^2} = 5.09\,\frac{h\tau x}{d} = 5.09\,\frac{27(64)(0.2)}{18} = 98 \text{ psi}$$

The predicted value of 98 psi is 6.7 percent lower than the measured value of 105 si given in Special Report 46 (1).

xample 3

The following hypothetical problem can be solved. It is necessary to design a flexle pavement on a cohesive subgrade with an unconfined compressive strength of 64 si. A certain design criteria states that the desired pavement section must be able support a rigid bearing plate of 24-in. diameter under an applied pressure, p, of 136 i such that the surface deflection does not exceed 0.4 in. Determine the minimum vement section.

Solution:

$$h = \frac{F}{4d\tau x} = \frac{p\,\pi d^2}{4d\tau x(4)} = \frac{pd}{5.09\tau x} = \frac{136(24)}{5.09(64)(0.4)} = 25 \text{ in.}$$

The field tests indicate a pavement thickness of 24 in. Thus, the predicted value higher by approximately 4.2 percent.

The preceding examples illustrate some possible applications of the results develed in this paper. It may be possible to use these results, or other results develop- by the methods presented, as a basis for a design criteria for flexible pavements. is important to point out that the present results indicate that the load-deflection aracteristics of flexible pavements are dependent on the total thickness of the section d not on the ratio of asphaltic concrete surface course to aggregate subbase. From viewpoint of riding characteristics and durability, under both normal wear and the verse conditions of water and frost action, the thickness of the surface course will quite important.

CONCLUSIONS

Non-dimensional techniques based on the methods of dimensional analysis seem to ovide a rational basis for analyzing rigid plate bearing tests on flexible pavements.

The test data reported by Benkelman and Williams (1, Table 4) has been successfulanalyzed by such techniques. The surface deflection, x, in inches can be expressed equational form as a function of the applied load, F, bearing plate diameter, d, pavent thickness, h, and the unconfined compressive strength, τ, of the subgrade in the lowing form:

$$x = \frac{F}{4dh\tau}$$

Several illustrative examples have been presented using this equation to indicate its sible application. Because of the test procedure used in the Hybla Valley Study this ation is restricted to surface deflections of approximately ½ in. for flexible pavents on cohesive subgrades. A significant result of the analysis is that the loadrying capacity of the flexible pavement as expressed by the surface deflection is dedent on the total pavement thickness and not on its proportion of asphaltic concrete subbase. With regard to the durability of the pavement the thickness of the asphaltic crete would be important.

The results also indicate that it may be possible to use the non-dimensional method in conjunction with durability studies to develop design criteria for flexible pavements. The authors recommend that additional field studies be conducted, using load creep procedures with greater surface deflections, on flexible pavement sections supported by subgrades of different unconfined compressive strengths subjected to various environmental conditions.

This study and other studies conducted by the senior author (11, 12, 13, 14, 15) indicate that both model and prototype research investigations designed and conducted on the basis of non-dimensional techniques can help prevent unnecessary duplication of costly, time-consuming experimental work. Many times, tests which seem to be different because of different values of the physical quantities involved, are in reality duplicate tests giving the same results when examined in non-dimensional form. The reason for this is that in the search for an explicit relation expressing a physical phenomenon, it is the values of the non-dimensional parameters, which are the new variabl[es] that are important and not simply the magnitudes of the individual physical quantities. Thus it is felt that if such a program is designed and conducted on the basis of non-dimensional techniques, there is a better chance of developing rational design criteria with a minimum of expended effort.

Although the method of analysis is a general research tool and some recommendations are made for future work, the quantitative results of this paper were obtained solely from the results of the cooperative study of flexible pavements conducted at Hybla Valley.

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