ACCELERATED TRAFFIC TESTS AT EGLIN FIELD, FLORIDA[1]

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SYNOPSIS

Several accelerated traffic test studies have been conducted by the Corps of Engineers to secure flexible pavement design data for airfield pavements. The accelerated traffic tests at Eglin Field, Florida, furnished such data on flexible pavements placed on uniform free draining sand subgrades where low ground water conditions prevailed. The test track included sections of an existing airfield pavement consisting of sand asphalt, sections of asphaltic concrete surfacing on variable thickness, and variable quality of sand clay base course materials, and similar sections of sand clay base course material without a wearing course. Large earth moving vehicles were used to simulate airplane traffic, and tests were performed for 15,000, 37,000, and 50,000 lb. wheel loads. Separate test lanes were provided for each wheel load. Each wheel load test was continued until 3500 coverages of a strip three times the tire contact width had been made. The behavior of all test sections was closely observed and measurement of resulting deformations was made frequently during the tests.

The results of the Eglin tests have in general verified the California Bearing Ratio method of design as now used by the Engineer Department when applied to flexible pavements placed on sand subgrades. They have indicated the desirability of obtaining a high degree of compaction in cohesionless subgrades and earth type base courses during construction, in order to prevent excessive rutting and roughening of the pavement surface due to traffic compaction of the subgrade. The performance record of unpaved earth type base courses, as observed in this study, indicated they are not satisfactory for moderate airplane traffic.

Accelerated traffic tests are helpful in supplying information on the design of pavements which are required to support the heavy wheel loads of military airplanes. These tests are a substitution for actual traffic experience. The study conducted by the Corps of Engineers at Eglin Field, Florida[2] was performed for the purpose of obtaining data and information on flexible pavement types common to the Gulf Coast areas and adjoining regions. Specifically, information was desired on the following:

a. Effectiveness of existing high quality sand asphalt pavement in protecting the sand subgrade against failure by lateral shear deformation when subjected to traffic with wheel loads up to 50,000 lb.

b. Comparative effectiveness, as pavement, of 1½ in. and 3 in. of asphaltic concrete on sand clay base courses and also the relative effectiveness of asphalt and tar surface primes on similar base courses.

c. To determine the minimum California Bearing Ratio (CBR) of base course material on a nonplastic and free draining sand subgrade that should be specified directly under high quality asphaltic concrete pavements 1½ and 3 in. thick for 15,000-, 37,000-, and 50,000-lb. wheel loads.

d. To determine what modifications, if any, are necessary in the tentative CBR design curves[3] and testing procedure for a nonplastic and free draining sand subgrade.

Sand asphalt pavements, as developed by the Florida Roads Department consist of subgrade sand mixed with 4.0 to 5.5 gal., usually RC-1 asphalt, per sqyd. per 6 in. depth. The cleaner subgrade sands generally require

[1] A complete report on these tests has been published by the War Department under same subject.

[2] A brief summary of the Eglin tests was given by T. A. Middlebrooks and R. M. Haines in "Results of Accelerated Traffic Tests of Runway Pavements," *Proceedings*, Highway Research Board, Vol. 23, p. 105 (1943).

[3] See "Soil Tests for Design of Runway Pavements," by T. A. Middlebrooks and Capt. G. E. Bertram, *Proceedings* Highway Research Board, Vol. 22, p. 162 (1942).

a stabilizer admixture of fines. The asphalt mixing is accomplished by a travelling plant pug-mill type mixer. After one to three weeks of manipulation and aeration, the sand asphalt mixture is compacted in thin layers by traffic rollers and the top lift is smoothed off with a tandem or a three wheel roller. These pavements average about 6 in. thick and over 3,000,000 sq. yd. have been constructed at Eglin Field, with highly satisfactory per-

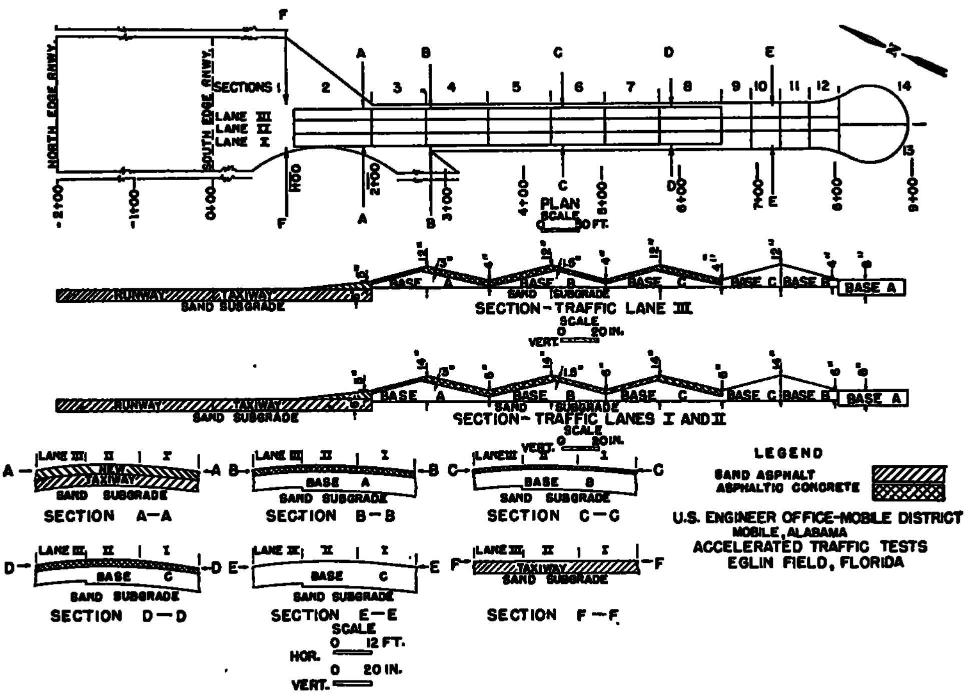


Figure 1. Plan and Profiles for Test Track

formance results for the past three years under airplane traffic up to 75,000 lb. gross weight.

Another principal flexible pavement design employed in the Gulf Coast areas consists of a sand clay base course with a bituminous surface treatment for light highway traffic or a thicker asphaltic concrete wearing surface for heavier traffic. Sand clay specifications require the soil binder (the portion passing a 10 sieve) to have from 35 to 85 per cent passing a No. 40 sieve, 65 to 90 per cent total sand, and from 9 to 23 per cent clay, with a liquid limit not greater than 25 per cent and a plasticity index not greater than 6 per cent.

Construction of a test track 700 ft. long and 50 ft. wide, with added turnarounds on a free-draining sand subgrade was started in March 1943 at one of the auxiliary fields at Eglin Field. The track was completed in May 1943. Three sand clay base course materials from the vicinity were selected on the basis of CBR values of approximately 80 per cent, 60 per cent, and 40 per cent when compacted at optimum moisture to 95 per cent of maximum density (Modified AASHO procedure). These were designated base A, B, and C, respectively. While considerable care was taken to secure the desired CBR variation in the three sand clay base course materials, the added compactive effort of the priming, surfacing, and traffic operations resulted in ultimate CBR values generally higher than intended. Figure 1 shows a plan and profile of the test track. It can be noted that with the exception of the turnarounds, the traffic was divided into separate lanes for each wheel load test. Figure 2 is a photograph of the completed test track.

The test track layout included fourteen test sections as follows:

Section 1 consisted of an existing sand asphalt pavement 6 in. thick constructed directly on the sand subgrade in 1941. No test lanes were established in this section.

Section 2 consisted of a sand asphalt pavement ranging in thickness from 6 to 11 in. The lower 6 in. was an existing pavement constructed in 1942. The thickness range was achieved by placing new sand asphalt pavement varying from 0 to 5 in. thick to increase the total thickness to 11 in.

Section 3 consisted of 1½ in. of asphaltic concrete laid on a variable thickness of base A material. The combined base and pavement thickness varied from 6 to 14 in. for test lanes I and II (37,000- and 50,000-lb. wheel loads) and from 4 to 12 in. for lane III (15,000-lb. wheel load).

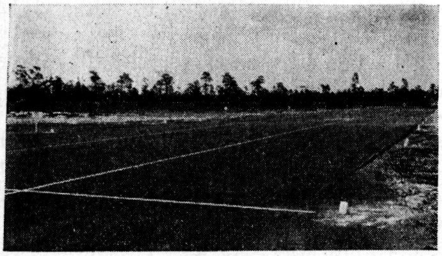


Figure 2. Completed test track before starting traffic tests

Section 4 consisted of 3 in. of asphaltic concrete laid on a variable thickness of base A material. The combined base and pavement thickness varied as in section 3.

Section 5 consisted of 3 in. of asphaltic concrete laid on a variable thickness of base B material. The combined base and pavement thickness varied as in section 3.

Section 6 consisted of 1½ in. of asphaltic concrete laid on a variable thickness of base B material. The combined base and pavement thickness varied as in section 3.

Section 7 consisted of 1½ in. of asphaltic concrete laid on a variable thickness of base C material. The combined base and pavement thickness varied as in section 3.

Section 8 consisted of 3 in. of asphaltic concrete laid on a variable thickness of base C material. The combined base and pavement thickness varied as in section 3.

Section 9 consisted of asphalt prime on a variable thickness of base C material. The base thickness varied from 6 to 10 in. for test lanes I and II and from 4 to 8 in. for test lane III.

Section 10 consisted of a tar prime only on a variable thickness of base C material. The base thickness varied from 10 to 14 in. for test lanes I and II and from 8 to 12 in. for test lane III.

Section 11 consisted of a tar prime on a variable thickness of base B material. The base thickness varied as in section 10.

Section 12 consisted of an asphalt prime on a variable thickness of base B material. The base thickness varied as in section 9.

Section 13 consisted of a tar prime only on a uniform 8-in. thickness of base A material and is the right half of the south turnaround.

Section 14 consisted of an asphalt prime only on a uniform 8-in. thickness of base A material and is the left half of the south turnaround.

Accelerated traffic tests were conducted during May, June, and July 1943, using 15,000-lb., 37,000-lb., and 50,000-lb. wheel loads. The 15,000-lb. wheel load traffic test was conducted with a 12-cu. yd. capacity LeTourneau Carryall pulled by a truck trailer power unit. The 37,000-lb. and 50,000-lb. wheel load traffic tests were conducted with a self propelled LeTourneau Model "A" Tournapull combined with model "NU" Carryall having a 32 cu. yd. struck capacity. Both units were loaded with sand. The wheels on each were spaced on the axles so that each trip covered a strip equal to 1½ times the tire contact width. Two trips of the equipment in each case constituted one complete coverage of a traffic trail three times the width of the tire contact. Each wheel load test was continued until 3500 coverages had been made. Figure 3 is a photograph of the Carryall, and Figure 4 is a photograph of the Tournapull fully loaded. The wheel loads, tire contact areas, and pressures for each traffic test are given in Table 1.

During the traffic tests record of the number of coverages in each lane was kept by an automatic traffic counter. Measurements of permanent deformations were made at frequent intervals during the tests. The method consisted in stretching a piano wire under 30-lb. tension pull across the track between two permanent bench marks for a reference base line. The vertical distance between this wire and the test track surface was measured at 6-in. intervals across the traffic trail with a steel rule graduated to 1/64 in. Twenty-five of these reference base lines were established at selected points across the test

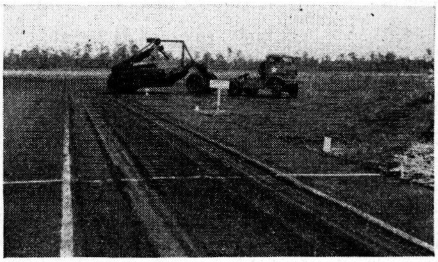


Figure 3. View showing difference in behavior of sand asphalt and asphaltic concrete in foreground after 170 coverages of 15,000-lb. equipment shown in background.

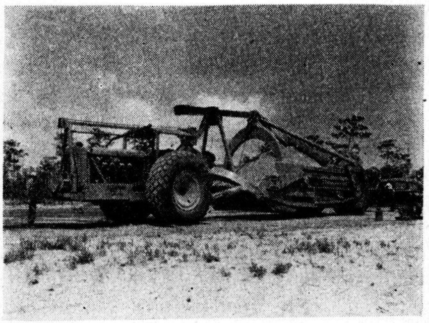


Figure 4. Tournapull used for 37,000- and 50,000-lb. traffic tests

TABLE 1

Type of Data	50,000-lb. Wheel Load	37,000-lb. Wheel Load	15,000-lb. Wheel Load
Wheel load front, lb.	50,000	37,000	13,500
Wheel load rear, lb.	54,000	37,000	16,500
Contact area front, sq. in.	719	570	230
Contact area rear, sq. in.	840	570	268
Contact pressure front, psi.	70	65	59
Contact pressure rear, psi.	64	65	62
Inflation pressure front, psi.	53	48	45
Inflation pressure rear, psi.	48	45	45

track. The results of the deformation data are summarized in Table 2, and are plotted to semilog scale on Figures 5, 6, and 7. In addition to the deformation measurements, visual observations of the test track under traffic were noted. The more important of these observations are listed as follows:

a. A measurable amount of permanent deformation was noted in all test sections after a few coverages had been completed. This was true for all three wheel loads.

b. After a few coverages had been completed, springing of minor proportions occurred in all traffic lanes where the paved and unpaved sections joined. After 500 coverages were completed, springing was noted at the low points between sections 6 and 7 in test lane III. In the thinnest part of the new sand asphalt, springing was noted after 100 coverages in test lane II. Springing continued throughout the test but did not increase in magnitude with the application of additional coverages. In general, the amount of springing was regarded as insignificant.

c. The primed sections 9, 10, 11, and 12 appeared to fail from excessive surface wear, due to the abrasive action of traffic after a low number of coverages for all wheel loads. See Figure 8.

d. Small to medium longitudinal hair cracks occurred along the edges of all test lanes of the asphaltic concrete sections; some of them along the edge of the 50,000-lb. lane developed into wider cracks. No cracks were observed in the sand asphalt sections.

8. After the completion of the traffic tests a thorough study of the pavement, base, and subgrade was made for the purpose of obtaining information on moisture conditions, densities, and in-place CBR values. A series of 14 trenches cut across the test track at 14 selected reference lines was used for inspection purposes and to measure accurately the final profiles of the pavement, base, and subgrade. In addition, an extensive laboratory study was conducted on undisturbed samples of the pavement, base, and subgrade. The results of the moisture, density, and CBR tests on the subgrade sand are summarized in Table 3. The results of similar tests on the various base course materials are summarized in Table 4, while tests on the pavement samples are summarized in Table 5. The mechanical composition of the subgrade and various base course materials are plotted on Figure 9.

The analysis of the results of this investigation depends to a large degree on whether the

TABLE 2
SUMMARY OF DEFORMATION PRODUCED BY TRAFFIC TESTS

Test Section	Reference Line No.	Pavement Thickness, in.	Base Thickness, in. — 15,000-Lb. Load	Base Thickness, in. — 37,000 & 50,000-Lb. Load	Total Thickness, in. — 15,000-Lb. Load	Total Thickness, in. — 37,000 & 50,000-Lb. Load	Base Material	Maximum Deformation, in. — 15,000-Lb. Wheel Load — Coverages 350	15,000-Lb. Wheel Load — Coverages 3,500	37,000-Lb. Wheel Load — Coverages 350	37,000-Lb. Wheel Load — Coverages 3,500	50,000-Lb. Wheel Load — Coverages 350	50,000-Lb. Wheel Load — Coverages 3,500
Sand Asphalt													
1	1	6.25	None		6.25	6.25	None	0.7	1.7	3.0	5.3	3.7	5.5
2	2	8.75	"		8.75	8.75	"	0.6	1.2	1.8	3.8	2.1	3.7
2	3	10.75	"		10.75	10.75	"	0.7	1.8	1.5	3.0	1.5	3.1
Asphaltic Concrete													
3	4	1.5	3.6	5.6	5.1	7.1	A	0.4	1.0	1.3	2.3	1.7	3.0
6	15	1.5	4.1	6.1	5.6	7.6	B	1.3	2.2	3.2	4.7	3.6	5.2
7	16	1.5	3.6	5.6	5.1	7.1	C	0.9	1.7	3.8	5.7	3.6	5.2
3	5	1.5	6.5	8.5	8.0	10.0	A	0.6	1.0	1.7	2.7	1.4	2.5
6	14	1.5	6.5	8.5	8.0	10.0	B	0.6	1.1	2.7	4.0	2.6	3.8
7	17	1.5	6.5	8.5	8.0	10.0	C	0.6	1.0	3.1	4.7	2.9	4.9
3	6	1.5	9.4	11.4	10.9	12.9	A	0.4	0.7	1.3	2.2	1.3	2.3
6	13	1.5	8.8	10.8	10.3	12.3	B	1.1	1.9	2.4	3.7	3.2	4.7
7	18	1.5	8.8	10.8	10.3	12.3	C	0.8	1.5	2.6	3.8	2.6	3.9
4	9	3.0	2.0	4.0	5.0	7.0	A	0.3	0.5	2.6	3.9	1.5	2.8
5	10	3.0	2.0	4.0	5.0	7.0	B	0.3	0.6	1.7	2.8	1.3	2.4
8	21	3.0	2.0	4.0	5.0	7.0	C	1.8	2.0	3.7	5.2	3.6	5.3
4	8	3.0	5.0	7.0	8.0	10.0	A	0.3	0.5	1.3	2.2	1.8	3.2
5	11	3.0	5.0	7.0	8.0	10.0	B	0.4	0.8	1.0	1.9	2.8	4.3
8	20	3.0	5.0	7.0	8.0	10.0	C	0.6	1.2	3.2	4.9	3.3	4.7
4	7	3.0	8.0	10.0	11.0	13.0	A	0.3	0.4	1.4	2.1	1.1	2.0
5	12	3.0	8.0	10.0	11.0	13.0	B	0.4	0.8	1.4	2.4	2.4	3.7
8	19	3.0	7.5	9.5	10.5	12.5	C	1.0	1.7	2.8	4.4	2.9	4.2
Primed Section													
9	22	0.0	5.0	7.0	5.0	7.0	C	2.5	5.7	5.2		5.8	
12	25	0.0	5.0	7.0	5.0	7.0	B	0.8		3.8			
10	23	0.0	11.0	13.0	11.0	13.0	C	1.0	3.2	2.7	4.2	3.4	5.7
11	24	0.0	11.0	13.0	11.0	13.0	B	0.8	2.3	2.0	3.9	3.6	4.7

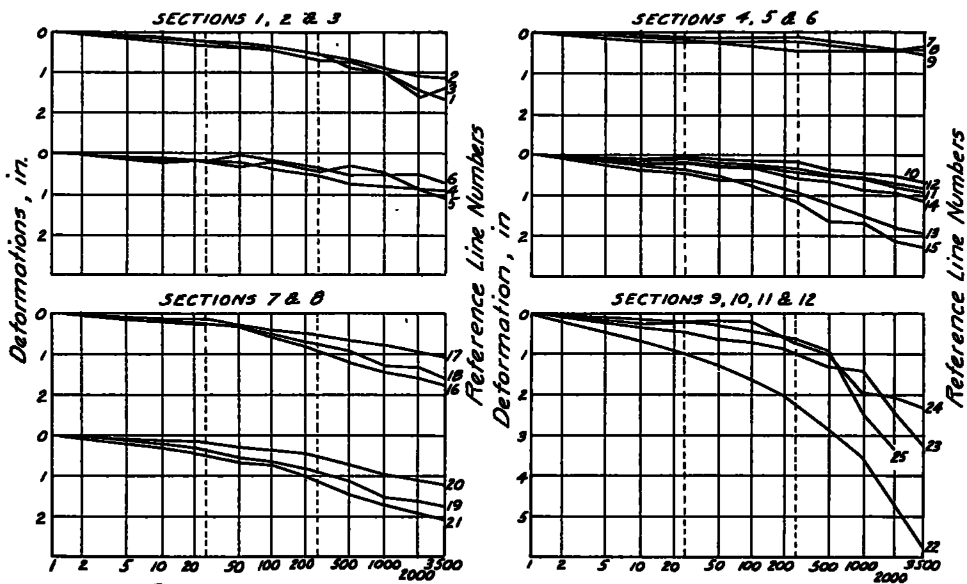


Figure 5. Permanent Deformation versus Coverages 15,000-lb. Wheel Load

deformations produced by the traffic were due to compaction only, to shear movements only, or to a combination of both compaction and shear. The California Bearing Ratio method is predicated on failure by shearing stresses.

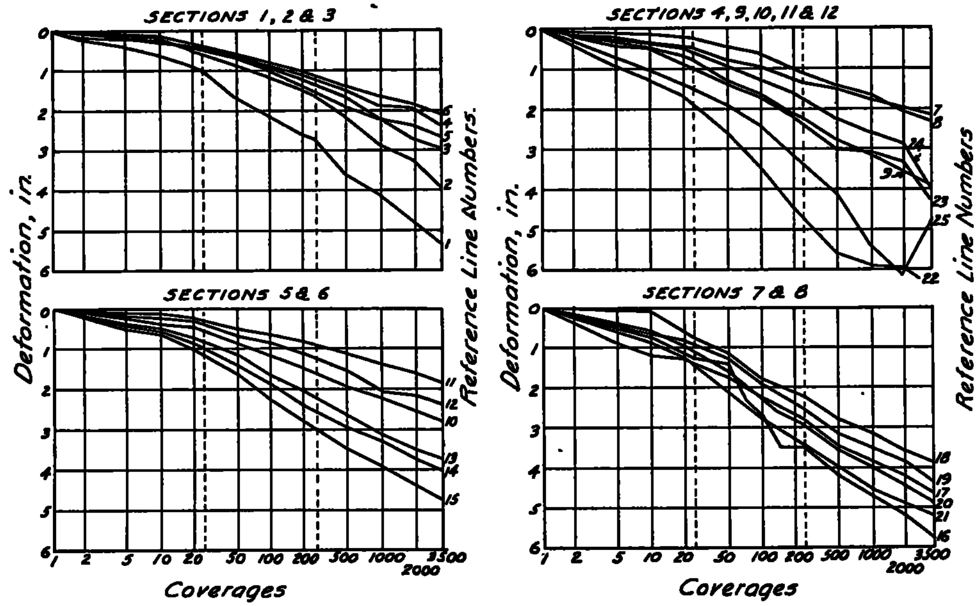


Figure 6. Pavement Deformation versus Coverages, 37,000-lb. Wheel Load

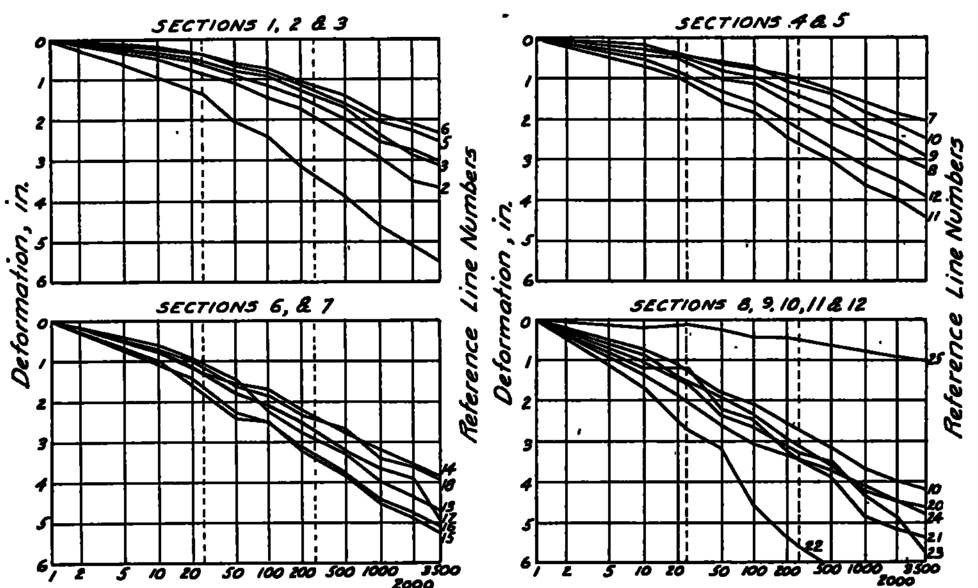


Figure 7. Permanent Deformation versus Coverages, 50,000-lb. Wheel Load

Such failure is generally recognized by noticeable springing or weaving of the surface and rapid increases in deformations and pavement disintegration under traffic. Observations of the behavior of the test track under traffic indicated springing of minor proportions at a few areas. The limited extent of this springing and the absence of progressive action indicate that only a small percentage of the measured deformations was caused by shear movements. Again such movements are ordinarily accompanied by appreciable uplifting in areas adjacent to the traffic line. Figure 10 shows the trench profiles for reference lines 7, 9, 10, and 12 as typical examples of these data. These show that the top of the pavement after traffic tests is generally at or below the elevation of the original top of the pavement. A study of the end areas of all 14 trench profiles above and below the original pavement elevation indicates that approximately 5 per cent was above and 95 per cent was below the original pavement elevation. If no compaction had occurred, these areas should have been about equally divided above and below the original pavement elevation. Thus, most of the deformations produced by the traffic must be due to compaction in the

Fig. 8. View of primed sections 9 and 10 after 380 coverages of the 37,000-lb. wheel load.

TABLE 3
SUBGRADE MOISTURE, DENSITY AND CBR DATA

Test Pit No.	Station	Ref. Line No.	Traffic Lane	Wheel Load, Lb.	Total Base & Pavement Thickness, in.	Depth Below Bottom of Base, in.	In-Place Moisture, per cent	In-Place Density, lb. per cu. ft.	Modified Dry Density, lb. per cu. ft.	Modified Density, per cent	Surcharge, lb.	In-Place CBR[a]	Undisturbed Laboratory CBR[a]
							INSIDE TRAFFIC LANES						
2	1 + 05	1	I	37,000	5.0	6	6.6	113.1	112.6	100.4	12.5	14	42
4	2 + 65	6	I	37,000	15.0	12	6.6	110.0	112.6	97.7	17.5	45	31
5	2 + 65	6	II	50,000	12.7	8	4.5	109.7	112.6	97.4	17.5	45	42
6	1 + 05	1	II	50,000	5.8	9	9.3	112.7	112.6	100.1	17.5	25	43
8	1 + 05	1	III	15,000	5.5	6	4.6	107.4	112.6	95.4	17.5	40	28
10	2 + 65	6	III	15,000	11.8	7	3.3	112.3	112.6	99.7	17.5	35	59
12	3 + 95	11	II	50,000	10.0	6	4.4	106.0	112.6	94.1	17.5	41	31
14	4 + 25	12	I	37,000	14.2	6	5.2	106.8	112.6	94.8	12.5	34	43
16	4 + 50	13	I	37,000	12.0	6	4.0	106.9	112.6	94.9	12.5	50	43
18	6 + 15	20	III	15,000	11.0	7	4.4	108.0	112.6	95.9	17.5		38
19	6 + 15	20	I	37,000	13.0	7	5.6	110.7	112.6	98.3	12.5	31	53
21	5 + 15	16	II	50,000	10.0	5	5.9	110.3	112.6	97.8	12.5	26	37
24	5 + 60	18	I	37,000	14.8	5	4.8	107.2	112.6	95.2			
26	5 + 90	19	II	50,000	15.2	6	4.4	112.7	112.6	100.1			
28	5 + 60	18	III	15,000	16.7	6	7.6	114.8	112.6	102.0			
29	4 + 25	12	III	15,000	8.5	6	4.5	113.9	112.6	101.2			
32	4 + 90	15	II	50,000	9.5	6	4.2	111.3	112.6	98.8			
Average							5.3	110.2	112.6	97.9		35	41
							OUTSIDE TRAFFIC LANES						
1	1 + 05	1	I	37,000	5.8	6	4.4	107.0	112.6	95.0	12.5	25	20
3	2 + 65	6	I	37,000	13.9	18	5.8	110.9	112.6	98.5	17.5	22	30
7	1 + 05	1	III	15,000	5.5	12	3.1	106.4	112.6	94.5	17.5	29	26
9	2 + 65	6	III	15,000	13.7	6	2.4	113.1	112.6	100.4	17.5	34	65
11	3 + 95	11	II	50,000	10.0	6	4.6	99.4	112.6	88.3	12.5	42	35
13	4 + 25	12	I	37,000	15.2	6	4.9	104.8	112.6	93.1	12.5	30	27
15	4 + 50	13	I	37,000	12.5	6	6.2	107.8	112.6	95.7	12.5	37	21
17	6 + 15	20	III	15,000	11.0	6	4.7	108.2	112.6	96.1	17.5		47
20	6 + 15	20	I	37,000	13.0	5	6.5	109.3	112.6	97.1	12.5	23	43
22	5 + 15	16	II	50,000	11.0	6	6.2	107.5	112.6	95.5	17.5	32	32
23	5 + 60	18	I	37,000	15.5	7	3.6	113.8	112.6	101.1			
25	5 + 90	19	II	50,000	15.5	6	5.3	111.5	112.6	99.0			
27	5 + 60	18	III	15,000	16.0	6	3.6	115.2	112.6	102.3			
30	4 + 25	12	III	15,000	10.2	6	4.4	112.1	112.6	99.6			
31	4 + 90	15	II	50,000	8.5	6	4.7	112.6	112.6	100.0			
Average							4.7	109.3	112.6	97.1		31	35

[a] Average of three individual tests.

pavement, base, or subgrade, either separately or in combination.

TABLE 4
BASE COURSE MOISTURE, DENSITY AND CBR DATA

Base Identification	Test Pit No.	Station	Ref. Line No.	Traffic Lane	Wheel Load, lb.	Pavement Thickness, in.	Base Thickness, in.	[a]In-Place Moisture, per cent	[a]In-Place Density, lb. per cu. ft.	Modified Dry Density, lb. per cu. ft.	Modified Density, per cent	In-Place CBR[a]	Undisturbed Laboratory CBR[a]
Inside Traffic Lanes													
A	4	2 + 65	6	I	37,000	2.0	13.0	10.3	119.9	123.7	96.9	71	45
A	5	2 + 65	6	II	50,000	1.7	11.0	7.2	116.7	123.7	94.3	77	41
A	10	2 + 65	6	III	15,000	1.8	10.0	5.6	115.6	123.7	93.5	53	69
Average							11.7	7.7	117.4	123.7	94.9	67	52
Outside Traffic Lanes													
A	3	2 + 65	6	I	37,000	1.9	12.0	7.5	118.7	123.7	96.0	48	23
A	9	2 + 65	6	III	15,000	1.7	12.0	6.9	117.8	123.7	95.2	57	63
Average							12.0	7.2	118.2	123.7	95.6	52	43
Inside Traffic Lanes													
B	12	3 + 95	11	II	50,000	3.5		5.4	118.9	126.2	94.2	75	58
B	14	4 + 25	12	I	37,000	3.2	11.0	6.5	124.2	126.2	98.4	122[b]	107
B	16	4 + 50	13	I	37,000	2.0	10.0	8.7	121.3	126.2	96.1	89	53
B	29	4 + 25	12	III	15,000	3.0	5.5	6.0	117.8	126.2	93.3	95	
B	32	4 + 90	15	II	50,000	1.5	8.0	5.4	121.9	126.2	96.6	78	
Average								6.4	120.8	126.2	95.7	86	73
Outside Traffic Lanes													
B	11	3 + 95	11	II	50,000	4.0		6.2	123.7	126.2	98.0	124[b]	45
B	15	4 + 50	13	I	37,000	2.5	10.0	5.8	123.5	126.2	97.9	37	72
B	13	4 + 25	12	I	37,000	3.25	12.0	7.2	114.8	126.2	91.0	53	48
B	30	4 + 25	12	III	15,000	3.25	7.0	6.5	114.5	126.2	90.7	58	
B	31	4 + 90	15	II	50,000	2.25	6.25	6.2	119.4	126.2	94.6	38	
Average								6.4	119.1	126.2	94.4	46	55
Inside Traffic Lanes													
C	18	6 + 15	20	III	15,000	3.2		4.6		121.7		72	28
C	19	6 + 15	20	I	37,000			4.8	119.9	121.7	98.5	66	11
C	21	5 + 15	16	II	50,000	2.0	8.0	5.3		121.7		49	
C	24	5 + 60	18	I	37,000	1.75	13.0	5.5	118.9	121.7	97.7	56	
C	26	5 + 90	19	II	50,000	3.25	12.0	5.3	121.7	121.7	100.0	60	
C	28	5 + 60	18	III	15,000	2.75	14.0	9.6	113.2	121.7	93.0	61	
Average								5.8	118.4	121.7	97.3	61	19
Outside Traffic Lanes													
C	17	6 + 15	20	III	15,000	3.5		4.9		121.7		46	36
C	20	6 + 15	20	I	37,000	3.0		5.3	117.8	121.7	96.8	30	21
C	22	5 + 15	16	II	50,000	2.0	9.0	5.8		121.7		54	
C	23	5 + 60	18	I	37,000	2.5	13.0	4.5	115.0	121.7	94.5	25	
C	25	5 + 90	19	II	50,000	3.5	12.0	6.1	116.2	121.7	95.5	21	
C	27	5 + 60	18	III	15,000	2.0	14.0	9.9	114.8	121.7	94.3	45	
Average								6.1	115.9	121.7	95.2	37	28

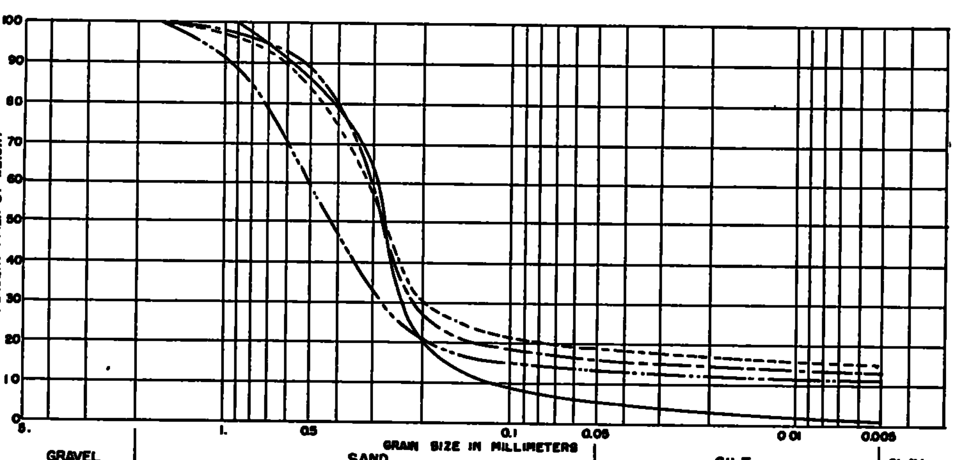
[a] Average of three individual tests.
[b] Not included in determining average.

side the trail. The difference amounting to about 2 per cent in the asphaltic concrete and

According to Table 5, a slight increase in the pavement density was found in the pavement samples taken directly from the traffic trail compared to those samples taken outside the trail. The difference amounting to about 3 per cent in the sand asphalt is almost insignificant when compared to the total deformation measured. According to Table 4, two of the base course materials showed a

TABLE 5
PAVEMENT TESTS

Test Pit No.	Station	Ref. Line No.	Traffic Lane	Wheel Load, lb.	Pavement Thickness, in.	Base Thickness, in.	Sieve Analysis—Per cent Passing: ¾ in.	⅜ in.	No. 4	No. 10	No. 40	No. 80	No. 200	Bitumen, %	Bulk Specific Gravity	Theoretical Density, %
ASPHALTIC CONCRETE																
INSIDE TRAFFIC LANES																
4	2 + 65	6	I	37,000	2.0	13.0	96	80	67	62	40	12	4	6.72	2.168	94.3
5	2 + 65	6	II	50,000	1.7	11.0	100	82	72	70	47	12	3	6.09	2.096	91.1
10	2 + 65	6	III	15,000	1.8	10.0	100	80	64	53	36	11	4	6.72	2.207	96.0
12	3 + 95	11	II	50,000	3.5		100	78	53	51	31	8	2	6.07	2.206	95.9
14	4 + 25	12	I	37,000	3.2	11.0	97	82	69	65	43	12	4	7.07	2.133	92.7
16	4 + 50	13	I	37,000	2.0	10.0	100	82	72	69	48	11	4	6.38	2.224	96.7
18	6 + 15	20	III	15,000	3.2		99	77	64	60	42	13	3	6.16	2.288	99.5
19	6 + 15	20	I	37,000	3.2		100	71	59	56	38	11	4	5.82	2.190	95.2
21	5 + 15	16	II	50,000	2.0	8.0	100	83	73	60	48	13	3	6.56	2.089	90.8
Average							99	79	66	61	42	11	3	6.60	2.180	94.4
OUTSIDE TRAFFIC LANES																
3	2 + 65	6	I	37,000	1.9	12.0	96	80	69	65	42	12	4	5.82	2.140	93.0
9	2 + 65	6	III	15,000	1.7	12.0	98	84	78	76	49	13	2	6.50	2.134	92.8
11	3 + 95	11	II	50,000	4.0		100	86	73	69	41	13	4	6.77	2.167	94.2
13	4 + 25	12	I	37,000	3.2	12.0	100	78	64	60	41	13	4	5.98	2.108	91.6
15	4 + 50	13	I	37,000	2.5	10.0	100	80	77	75	48	14	3	6.77	2.190	95.2
17	6 + 15	20	III	15,000	3.5		95	69	49	41	26	8	3	6.50	2.243	97.5
20	6 + 15	20	I	37,000	3.0		100	83	62	59	36	11	4	6.18	2.119	92.1
22	5 + 15	16	II	50,000	2.0	9.0	94	72	52	46	27	9	3	6.04	1.994	86.7
Average							98	79	66	60	39	12	3	6.42	2.137	92.9
SAND ASPHALT																
INSIDE TRAFFIC LANES																
2	1 + 05	1	I	37,000	5.0					100	68	10	3	4.60	1.988	80.8
6	1 + 05	1	II	50,000	5.8					100	69	12	5	5.15	1.920	78.1
8	1 + 05	1	III	15,000	5.5					100	68	12	5	4.49	1.896	77.1
Average							......	......	.....	100	68	11	4	4.78	1.935	78.7
OUTSIDE TRAFFIC LANES																
1	1 + 05	1	I	37,000	5.8					100	69	10	3	5.02	1.836	74.7
7	1 + 05	1	III	15,000	5.5					100	69	12	5	5.15	1.916	77.9
Average							....	.....	..	100	69	11	4	5.08	1.876	76.3



LEGEND
- - - - - - BASE A
——— ——— BASE B
——·——·—— BASE C
———————— SUBGRADE

MATERIAL	L.L.	P.I.	S.L.	S.R.
BASE A	22.	3	18.	1.68
BASE B	20	3	16	1.76
BASE C	23	3	20	1.64
SUBGRADE		0		

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MOBILE, ALABAMA.
ACCELERATED TRAFFIC TESTS
EGLIN FIELD, FLORIDA.

Figure 9. Mechanical Analysis of Base Courses and Subgrade

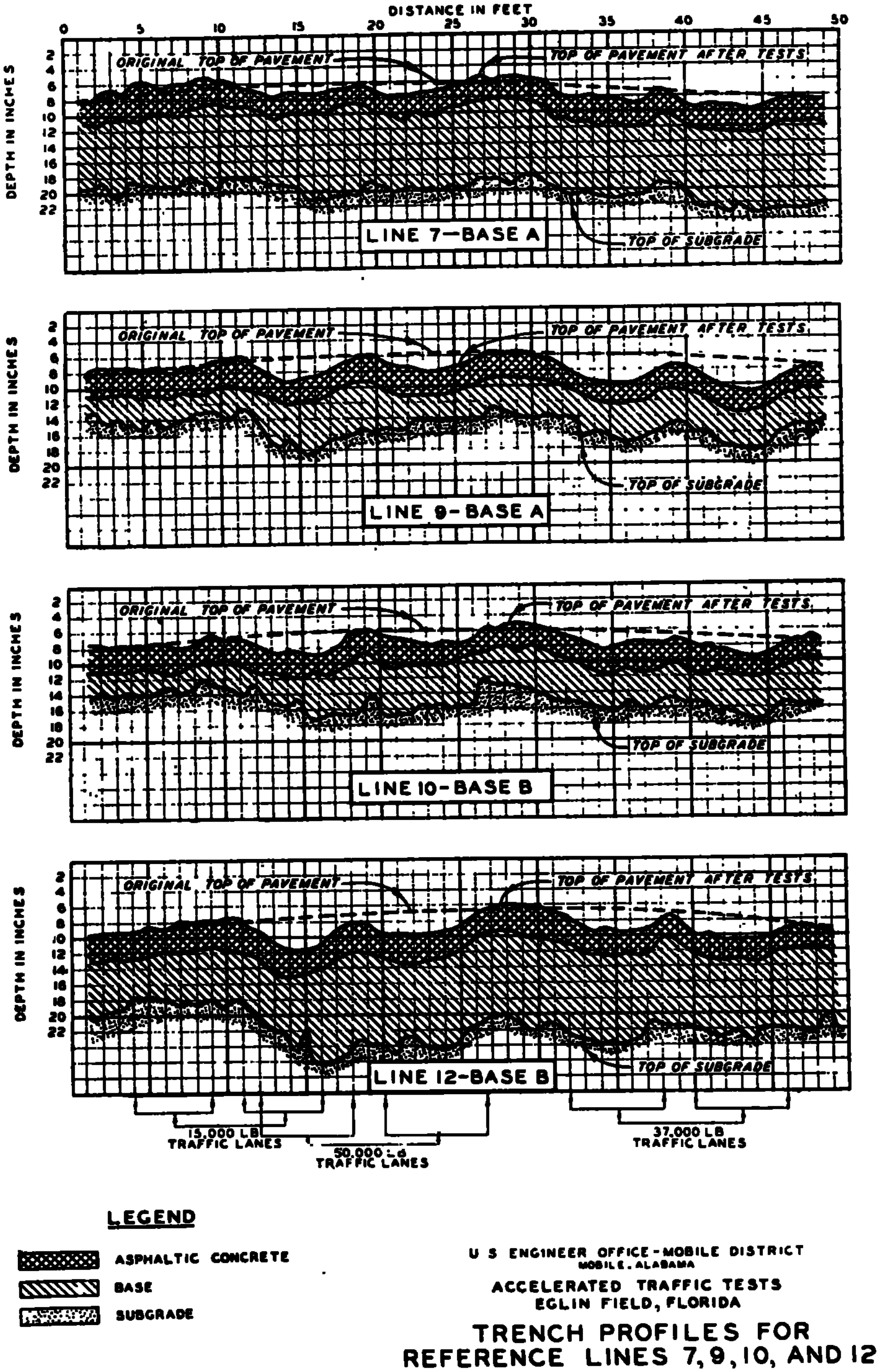

DISTANCE IN FEET
0 5 10 15 20 25 30 35 40 45 50
DEPTH IN INCHES
2 4 6 8 10 12 14 16 18 20 22
ORIGINAL TOP OF PAVEMENT
TOP OF PAVEMENT AFTER TESTS
TOP OF SUBGRADE
LINE 7—BASE A
LINE 9-BASE A
LINE 10-BASE B
LINE 12-BASE B
15,000 LB TRAFFIC LANES
50,000 LB TRAFFIC LANES
37,000 LB TRAFFIC LANES
LEGEND
ASPHALTIC CONCRETE
BASE
SUBGRADE
U S ENGINEER OFFICE-MOBILE DISTRICT
MOBILE, ALABAMA
ACCELERATED TRAFFIC TESTS
EGLIN FIELD, FLORIDA
TRENCH PROFILES FOR
REFERENCE LINES 7, 9, 10, AND 12

Figure 10

small increase in density for those portions immediately beneath the traffic trail against those portions outside the trail. This table and the trench profiles, Figure 10, indicate that the amount of compaction in the base course was very slight in comparison to the total amount measured. Consequently, the greater portion of the deformation must have occurred through compaction of the sand subgrade.

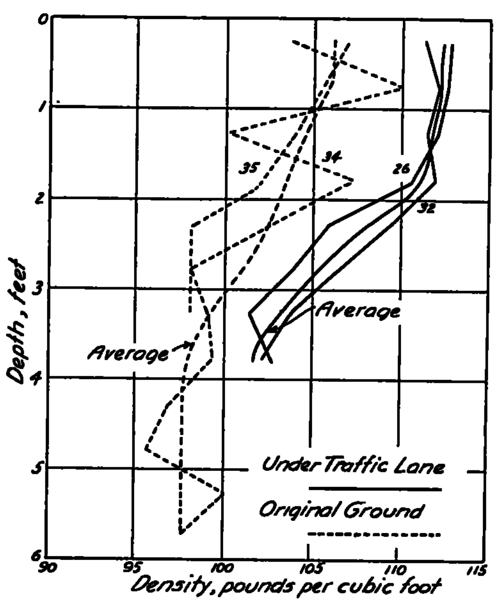


Figure 11. Subgrade Density Study. Each point plotted is the average of 3 test values. Numbers beside curves refer to test pits.

The test track construction procedure following the usual local practice, did not include special effort to compact the subgrade sand. The only construction compaction it received was that which resulted from the grading and paving operations. When it became apparent that the greater portion of measured deformation was due to consolidation of the subgrade, two test pits were put down in virgin ground several hundred feet from the test track and the densities were determined at 1-ft. intervals to a depth of 5 ft. Similar data were secured for the subgrade immediately beneath the 50,000-lb. wheel traffic trail of the test track. The results of these density determinations are shown on Figure 11. These data indicate that the top of the original ground has a density of about 6 lb. per cu. ft. less than that of the test track subgrade immediately beneath the base course, and about 4 lb. per cu. ft. less at a depth of about 4 ft. Some of this difference in density was produced by construction, and it is estimated that an average of about 50 per cent of the difference is due to this cause. This indicates that the subgrade compaction resulting from the vibration and load of the traffic must extend to a considerable depth to account for the measured deformations.

A complete review of all deformation-traffic data leads to the following observations:

a. More than half of the total deformation occurred prior to the completion of 350 coverages for all wheel loads.

b. The average deformations caused by the 37,000-lb. wheel load were only slightly less than those for the 50,000-lb. wheel load while the average deformations produced by the 15,000-lb. wheel load were about $\frac{1}{2}$ of those measured under the 37,000-lb. wheel load. Deformations measured in the sand asphalt sections were generally higher than deformations measured for equal thickness of sand clay base and asphaltic pavement.

c. In general, there was little or no appreciable difference in the deformations measured in the $1\frac{1}{2}$-in. thickness of asphaltic concrete and those measured in the 3 in. of asphaltic concrete section.

d. In general, an increase in the total thickness of base and pavement was accompanied by some reduction in the total deformation.

e. A plot of deformation versus number of coverages shows only two or three sharp increases in the rate of deformation with increased load repetitions. These are probably due to erroneous data.

f. The prime treated sections suffered early excessive wear from the abrasive action of traffic under all wheel loads. This rate of wear was such that the relative merits of asphalt and tar primes could not be determined.

One of the important factors affecting the behavior of the materials tested under the accelerated traffic wheel loads was the absence of excess water, due to the low groundwater table. This low water table was probably due to the uniform free-draining sand subgrade extending to great depths, and thus it was not possible for hydrostatic pore pressure to build up in the subgrade. This fact is substantiated by the data presented in this report which show (1) that no lateral shear deformation occurred, (2) the lack of pronounced springing, and (3) the ready consolidation of the sand subgrade.

In correlating the results of the traffic tests with capacity operation, it has been assumed that if no shear deformation occurred in the base or subgrade during 3500 coverages in the traffic test the design is satisfactory to insure no detrimental shear deformation during capacity operation. Since apparently no shear deformation occurred at the thin end of the section during the traffic test, the data indicate that a CBR value of 35 per cent in a clean well-drained sand subgrade covered with a total base and pavement thickness of 4 in. will not fail from shearing stresses under capacity operation of a 15,000-lb. wheel load. The data also indicate that the same material covered with a total base and pavement thickness of 6 in. will not fail from shearing stresses produced by capacity operation of 50,000-lb. wheel loads.

Table 4 indicates that the CBR values of the three base courses are 67, 86, and 61 for materials A, B, and C, respectively. Since no appreciable shear deformations occurred in any of the base materials for any of the wheel loads, it could be assumed that a sand clay base having a CBR value of 60 when placed on a well-drained, clean sand subgrade and surfaced with a high quality wearing course will be satisfactory for wheel load traffic up to 50,000 lb. Due to the very similar nature of the three base courses the range of CBR values originally intended was not attained.

In as much as the sand asphalt pavements showed no cracking or other indications of distress at the completion of the traffic runs, it may be assumed that a total thickness of 6 in. of high quality sand asphalt is satisfactory to prevent detrimental shear deformations in the pavement course for capacity operation of a 50,000-lb. wheel load. Similarly the results do not show that a 3-in. asphaltic concrete pavement is appreciably superior to a 1½-in. asphaltic concrete pavement for the conditions of the test as both showed no signs of distress other than permanent deformation under capacity operation of a 50,000-lb. wheel load. Visual observations of the sand asphalt turnaround showed that the pavement surface was scarred and pitted to an appreciable degree from the turning action of the traffic equipment. It is possible that had one of the turnarounds been asphaltic concrete pavement of variable thickness, a more definite benefit of the 3-in. asphaltic concrete pavement over the 1½-in. asphaltic concrete pavement would have been observed.

From the foregoing it is indicated that the present CBR tentative design curves for total base and pavement thickness on clean, well-drained sand subgrades are slightly on the conservative side in so far as preventing failure by shearing stresses is concerned. However, the deformations measured for the 37,000-lb. and the 50,000-lb. wheel loads are of such magnitude that hazard to fast moving aircraft may result. Consequently, design of pavements on loosely compacted sand subgrades must take into account some limiting deformation value. This limiting value must be established by the experience of the traffic using the pavement and can not be determined from the data secured in this investigation. The need for adequate compaction of these sand subgrades to depths sufficient to reduce such deformations is definitely indicated.

CONCLUSIONS

The conclusions derived from this investigation are based on the performance of the materials used and existing at this test section under the conditions prevailing at the time the tests were conducted. It should be emphasized that the subgrade was never in a saturated condition at any time during the testing period.

General

The following general conclusions are believed applicable to the design and construction of flexible type airfield pavements for capacity operation when the *subgrade is an unsaturated, noncohesive, and free-draining sand.*

a. A CBR value of 60 in sand clay bases over clean, well-drained sand subgrades, when surfaced with 1½ or 3 in. of asphaltic concrete, was satisfactory to prevent shear deformation in the base for capacity operation of all wheel loads up to 50,000 lb.

b. A thickness of pavement and base of 4 in. over an unsaturated, noncohesive, free-draining sand subgrade with a CBR of 35 is satisfactory to prevent detrimental shear deformation in the subgrade during capacity operation of a 15,000-lb. wheel load.

c. A thickness of pavement and base of 6 in. over an unsaturated, noncohesive, free-draining sand subgrade with a CBR of 35 is satisfactory to prevent detrimental shear deformation in the subgrade during capacity operation of 37,000- and 50,000-lb. wheel loads.

d. The total thicknesses of pavement and base required by the California method tentative design curves are somewhat conservative from the standpoint of preventing detrimental shear deformation in unsaturated, noncohesive sand subgrades with CBR values in the range of that of the subgrade tested.

e. The prevention of objectionable settlement due to consolidation should be a factor in the design of flexible pavements as well as the prevention of objectionable settlement due to shear deformation.

f. Sand subgrades similar to that at Eglin Field should be adequately compacted to sufficient depths to eliminate objectionable settlement under traffic. For heavy wheel loads compaction of the upper 2 ft. of the subgrade to approximately 100 per cent modified density, with some compaction to a depth possibly as deep as 6 ft., is indicated as necessary to prevent objectionable settlement under traffic.

g. Although the minimum thickness permitted by the design curves may be more than required to prevent shear deformation in this type of subgrade, a thickness greater than the minimum may be required to prevent objectionable settlement from traffic compaction, unless the upper portion of the subgrade is compacted to an exceptionally high and uniform density.

h. Tar and asphalt prime coats on sand-clay bases are not adequate as wearing courses for even limited operation of heavy wheel loads.

Specific

In addition to the general conclusions, the following conclusions concerning the materials and pavement sections tested are presented.

a. A thickness of 1½ in. of good quality asphaltic concrete was adequate as a wearing course over the sand-clay base with a CBR of 60 for all wheel loads in the traffic tests.

b. A thickness of 6 in. of mixed-in-place good quality sand asphalt (when properly cured) was adequate as a wearing course and was satisfactory to prevent shear deformation in the underlying unsaturated, noncohesive, free-draining sand with a CBR of 35 for all wheel loads in the traffic tests.

c. The greater portion of the permanent deformation produced in the pavement surface by the accelerated traffic was due to consolidation of the sand subgrade.

d. In general, the 1½ in. asphaltic concrete performed equally as well as the 3-in. over all base materials tested.

e. In general, there was a trend toward a reduction in settlement with an increase in thickness of any base.

f. There was a general trend toward a considerable reduction in settlement with an increase in thickness of sand asphalt.

g. The tar and asphalt prime coats can be considered satisfactory as a dust palliative only, provided that the surface is not subjected to traffic. Neither type of prime coat was satisfactory to resist the action of traffic. Due to rapid deterioration of both prime coats under traffic, no conclusions can be drawn as to the superiority of one over the other.

DISCUSSION

Mr. L. A. Palmer, *Bureau of Yards and Docks, Navy Department:* Mr. Robeson has made a very valuable contribution to our fund of information in the field of airport pavement design.

It is the opinion of this writer that accelerated traffic tests on relatively narrow widths, 1½ to 3 tire widths, may provide usable information concerning the relative merits of different materials and construction procedures. As a basis for design of pavement thickness however, the test is too far removed from reality to have much practical value. Traffic on runways is distributed. Over a long period a plot of the total number of landings and take-offs versus distance from positions of planes to the centerline will yield a curve very similar in shape to the probability curve with the maximum ordinate at the centerline. This fact could be utilized to good advantage in accelerated traffic tests. A test strip could be divided into several adjoining lanes and the traffic test would simulate traffic distribution on a typical runway on a reduced area scale. The center lane, say the middle third or fourth of the strip, would receive the most traffic and outward from this lane, the number of coverages would be reduced progressively. Of course it would be necessary to give all the lanes the proportionate number of coverages from the very outset. The results of a test so conducted could very well serve as a basis for the design of pavement thickness. Probably a very great number of coverages would not be necessary since Mr. Robeson reports that more than half of the total deformation was realized prior to the completion of 350 coverages for all wheel loads.

Conclusions concerning the respective merits in performance of 1½- and 3-in. asphaltic concrete surfacings seem hardly warranted. The slowly moving wheel loads in the tests at Eglin are not directly comparable to the action of swiftly moving plane wheels having tire pressures in excess of 85 lb. The surfacing must be designed for planes turning. It is true that planes are supposed to turn only at the ends of runways. They usually do so and it is considered to be poor airmanship to do otherwise but in design it must be expected that this restriction will be violated again and again. The tire pressures for the tests at Eglin varied from 45 to 53 lb. which is slightly less than half of the tire pressures of the Navy's carrier based planes. Even the slowly moving Carryall was observed at Eglin to scar and abrade the surface in making turns.

Mr. Robeson rightfully emphasizes *vibration* combined with load in effecting deep seated compaction of the sand subgrade. In a discussion of a paper by Professor Tschebotarioff dealing with vibration loads on sand and appearing elsewhere in this volume[1], this writer has presented data showing how the traffic of light planes compacted a sand subgrade below a 6-in. flexible pavement. After two years or more of traffic by light, single motor training planes, the density of the top foot of sand subgrade was 103 per cent, on the average, of the modified A.A.S.H.O. laboratory maximum density. The pavement at this field may have failed under very heavy (50,000 lb. or more) wheel loads at the time it was built and before it had carried traffic. Today, wheel loads of this order of magnitude could scarcely make a mark on it other than to mar the thin surfacing in making turns. One economical way to pave an airport on a sand subgrade would be to design it for light planes and let such planes compact the sand through the pavement until its bearing value is increased to an extent such that the field is safe for much heavier types of planes. The data are in the field, obtainable at existing airports and even the most skeptical will be convinced if he will investigate this possibility.

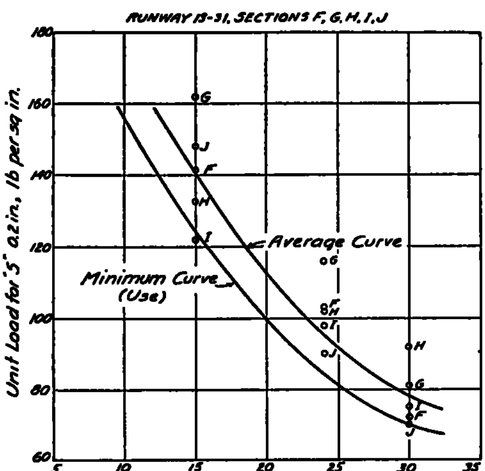
The important point to bear in mind is that in the case cited by this writer, Waldron Field, NATB, Corpus Christi, Texas, compaction of the sand subgrade was effected *generally* under the pavement and not in any local area. Repeated trips by one of the light planes over a narrow strip of 1½ tire widths, starting at the completion of paving, would have caused the pavement to fail.

At the present time data similar to those reported at Waldron Field are being obtained

[1] See page 423.

by this writer from other sources where there are sand subgrades.

Mr. Robeson states that for sand subgrades, loosely compacted, the design of pavement must take into account some limiting value of the vertical deformation or settlement. This is absolutely correct. The CBR test has great value as a test. Disagreement arises from the interpretations attached to it. If it can be imagined that it is a shear test, then surely there can still be no doubt that it is a poor second to the laboratory triaxial or even the box shear test. When



CONTACT PRESSURE lb per sq in.	DIAMETER in.	AREA sq in.	TOTAL LOAD lb
60	—	—	>50000
70	30	707	49500
80	25.5	509	40700
90	22.5	399	35900
100	20	314	31400
110	18	251	27600

Figure A

it is known that a CBR of 5 or 70 can be obtained with one and the same soil, depending solely on the method of compacting and molding the samples, then one can appreciate the possible errors in the interpretation of CBR test data. Using the extrapolated curves as prepared by the U.S.E.D., and considering a 75,000-lb. wheel load, the CBR value of 5 requires a pavement that is 38 in. thick whereas a pavement of 6-in. thickness is enough for the higher CBR value. In the use of these values in design, the fact that the CBR of the subgrade, subbase, base course etc., can and often does increase during traffic is apparently discounted. A condition of saturation is assumed to continue throughout the life of the pavement. Presumably then, surface drainage, although it is an expensive thing to obtain, is desirable only for the reason that it eliminates the hazard of "ground loops" when landing during a hard rain.

This writer has had laboratory tests made with undisturbed subgrade soils taken from under existing pavements at 15 different air stations, at least 10 samples at each field, and has found saturation of the subgrade in two instances, both being the ground immediately under the transverse expansion joints of concrete pavements where the seal had not been good.

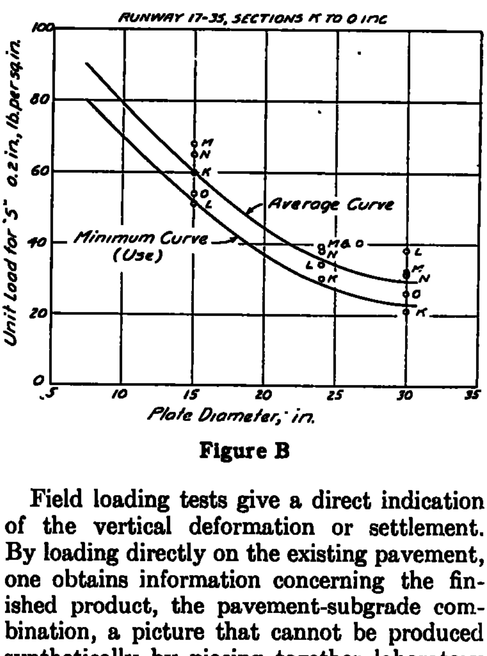


Figure B

Field loading tests give a direct indication of the vertical deformation or settlement. By loading directly on the existing pavement, one obtains information concerning the finished product, the pavement-subgrade combination, a picture that cannot be produced synthetically by piecing together laboratory test data obtained with the several materials. This writer has cooperated in the pavement evaluation at a considerable number of Naval Air Stations. The procedure as recommended and used by this writer is briefly the following: At least five test sections are selected on each runway. At each test section, three loading tests are made, each test being at least 10 ft. from the others. The three tests are made with three different sizes of steel plates; 15, 24 and 30 in. in diameter. The selected test sections are in the poorest runway areas. A subgrade soil survey is made prior to making the loading tests and on an area (it may be an entire runway) where the soil conditions are practically uniform, the loading test data may be averaged.

The data are plotted according to the scheme shown in Figures A and B. The unit load on the plate when (at equilibrium) the total plate deflection was 0.2 in. is plotted against the plate diameter. An average curve (see Figures A and B) is drawn but is not used in computations. A minimum curve, parallel to the average curve and passing through the minimum values on the ordinate scale, is drawn and used for computing the safe maximum wheel loads.

The tire foot-print area is assumed to be circular. With reference to Figure A, it is seen that for a tire contact pressure of 70 lb. per sq. in. the plate diameter, using the minimum curve, is 30 in., the diameter of a circle having an area of 707 sq. in. The product, $70 \times 707 = 49{,}500$ lb. to the nearest 500 lb. and is the safe wheel load for this contact pressure. The tire pressure is considered to be $\frac{70}{1.1} = 63.6$ lb. per sq. in.

According to this procedure, the maximum safe wheel load diminishes as the tire pressure increases, this conclusion following directly from test data although it is also theoretically sound. It would be most remarkable if this were not the case since by the Westergaard analysis it follows directly that the safe wheel load on a concrete pavement diminishes with increasing tire pressure, all other variables remaining fixed.

The matter of adequate surfacing is a consideration separate and apart from that of static wheel load bearing capacity in the pavement evaluation according to the foregoing procedure. In addition to adequate resistance to vertical deflection, the pavement must have sufficient scuffing resistance. The fact that surface reinforcement applied to the existing pavement will incidentally increase its bearing value is not, however, overlooked. A 20 per cent increase in bearing value has been observed in one instance when the 6-in. pavement (base course plus surfacing) received an additional 1½ hot-mix, hot-laid asphaltic concrete cover.