THE EVALUATION OF WHEEL LOAD BEARING CAPACITIES OF FLEXIBLE TYPES OF PAVEMENT[1]

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SYNOPSIS

The Bureau of Yards and Docks and the Bureau of Aeronautics of the Navy Department jointly sponsored an extensive investigation of concrete and bituminuous-surfaced airfield pavements at more than 40 Navy and Marine Corps air stations. This involved making more than 3,000 plate loading tests ranging up to 75,000-lb maximum loads, also taking and testing an approximately equal number of samples of subgrades and pavement-components.

The objectives of this investigation were: (1) to obtain directly the maximum safe wheel load capacities of the pavements at the specific fields; (2) to obtain basic information of value in estimating the wheel load capacities of pavements at other fields not included in the study; and (3) to obtain basic information of value in checking current design procedures and possibly developing new design procedures and methods of construction.

It is hoped that the analyses of the data will indicate the relative merits and limitations of different pavement design criteria and aid in an approach to more economical pavement design. This paper is a progress report of the program.

The general procedure for evaluating the wheel load bearing capacities of flexible types of pavements at naval air stations is described. The procedure includes plate loading tests, both on the pavement surface and on the subgrade, and in-place density and moisture tests of base course, sub base and subgrade materials. Laboratory tests include triaxial tests on undisturbed subgrade samples, density and moisture relations, Atterberg limits, grain size, asphalt stability and other tests. Comparisons are made between the in-place densities and moisture percentages of subgrade soils and the laboratory compacted maximum densities and optimum moistures of the same soils. The limitations and applicability of theory are being studied. Data indicate the relationship between bearing capacity and tire pressure and also the relationships between pavement thickness, base course density, and subgrade properties on the bearing value of the pavement.

A critical study is being made of certain yardsticks, used in pavement thickness designs, which involve the assumption that soft wet subgrades are normally to be expected under airport pavements in use. Thus far in the study, the actual subgrade conditions as found fail completely to bear out this assumption.

Five field testing units were organized as follows:

Unit No. 1—New England and North Atlantic States

Unit No 2—Middle and South Atlantic States

Unit No. 3—Great Lakes, Ohio River, and South Atlantic States

Unit No. 4—South Pacific Coast Stations

Unit No. 5—North Pacific Coast Stations

The organization engaged in field testing and sampling was comprised of five distinct units. Each unit was in charge of a commissioned officer who was assisted by other officers, enlisted personnel, and civilians ranging in number from 11 to 20 men. Each separate unit was assigned certain air stations within specific geographic divisions of the United States.

Each unit was equipped with three trailers each weighing 100,000 lb when fully loaded. A crew of three men was assigned to each trailer, and each crew made loading tests with a hydraulic jack with calibrated gauge, loading on steel plates of different sizes on the pavement and on a 30-in. diameter plate on the subgrade. Each unit had also a crew of from six to nine men who took subgrade and pavement samples, plotted load test data, prepared all drawings showing soil profiles and test loca-

[1] Progress report of a joint project of the Bureau of Yards and Docks and the Bureau of Aeronautics of the Navy Department.

tions, and who operated the mobile field testing laboratory.

The field laboratory personnel made tests required for subgrade classification, in-place density tests of base course, sub-base and subgrade materials, and also took undisturbed subgrade samples and samples of asphalt surfacing, base course, sub-base, and concrete pavement sections for shipment to laboratories of the Corps of Engineers, War Department.

Various laboratories of the Corps of Engineers, War Department, rendered very valuable assistance by making tests of asphalt surfacing, base course and sub-base materials, subgrade soils, and concrete which could only have been made at a central laboratory.

Prior to setting up the five mobile units a complete laboratory and field study of pavements had been made at eight Naval Air Stations at Corpus Christi, Texas and at six other fields at Pensacola, Fla. The general procedure followed during this earlier work was essentially the same as that later adopted by the mobile testing units. The tests and studies at Corpus Christi and Pensacola were made under the direction of the local Public Works Officers, assisted by station civilian and enlisted personnel.

TEST PROCEDURES

The general procedure of each mobile unit on arriving at an air station was to study the plans and specifications for the construction of the pavements, collect all pertinent data relating to the original design of the pavements and drainage system, study the records of the actual construction, and note any indicated improvements of pavements and drainage facilities. These studies were made concurrently with the subgrade soil surveys which provided the essential information required for selecting the sites for sampling and field tests.

There were two classes of test procedures, field and laboratory. The field procedures included first, sampling and loading tests on the pavement surface with three sizes of steel plates, 15, 24 and 30 in. in diameter. The location of sites for loading tests followed a general survey of the subgrade conditions. Loading test sites were then selected so as to have all different subgrade types, fill and cut sections, etc., represented. Figure 1 shows in plan the location of soil borings at a typical field. Soil borings were made to depths of 4 to 6 ft and disturbed samples were taken at 1-ft depth intervals. Figure 2 is a typical section showing the soil profile along the longitudinal centerline of a runway. The profiles were plotted for each runway and taxiway following simple soil classification tests in the field laboratory (Fig. 3). Figure 4 shows a typical location plan for the plate loading tests and a typical layout for a load test group including load tests with the three sizes of plates and sampling of pavement and subgrade. The several pavement components, asphalt surfacing, base and sub-base courses and undisturbed subgrade samples were all taken from the 3- by 3-ft section, located midway between the spots selected for loading with the 24- and 30-in. diameter plates. Sampling was done after completion of the loading tests. Figure 5 shows the type of trailer used for reaction in loading. Figure 6 shows a typical load test assembly in loading on the pavement surface. Figure 7 illustrates a load test on the subgrade with the 30-in. plate (the other plates, 15- and 24-in. diameter, were not used in loading on the subgrade). The load test on the subgrade was performed in exactly the same spot where loading with the 30-in. plate on the pavement surface was accomplished.

Loads were applied on the pavement with calibrated hydraulic jacks of 100,000-lb capacity in increments of 10 lb per sq in. on each plate size. Loads were allowed to act for a time such that either the rate of settlement, as observed by micrometer dials, became equal to or less than 0.002 in. per six min (an arbitrary figure) or until 30 min had elapsed, whichever occurred first, before adding the next load increment. The recovery after 30 min following the full release of the final load was measured in all cases. In loading on the pavement surface, the final load was either that required to produce 0.2 in. settlement (the figure first suggested by the Committee on Flexible Pavement Design of the Highway Research Board) or 75,000 lb, whichever was accomplished first. In loading on the subgrade the final load was that required to produce 0.2 in. settlement and load increments were usually 5 lb per sq in. or less, depending on the soil.

In-place density tests of base course (see Fig. 8) sub-base and the top 6 in. of subgrade were made with the sand method (*1*)[2]. Mois-

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

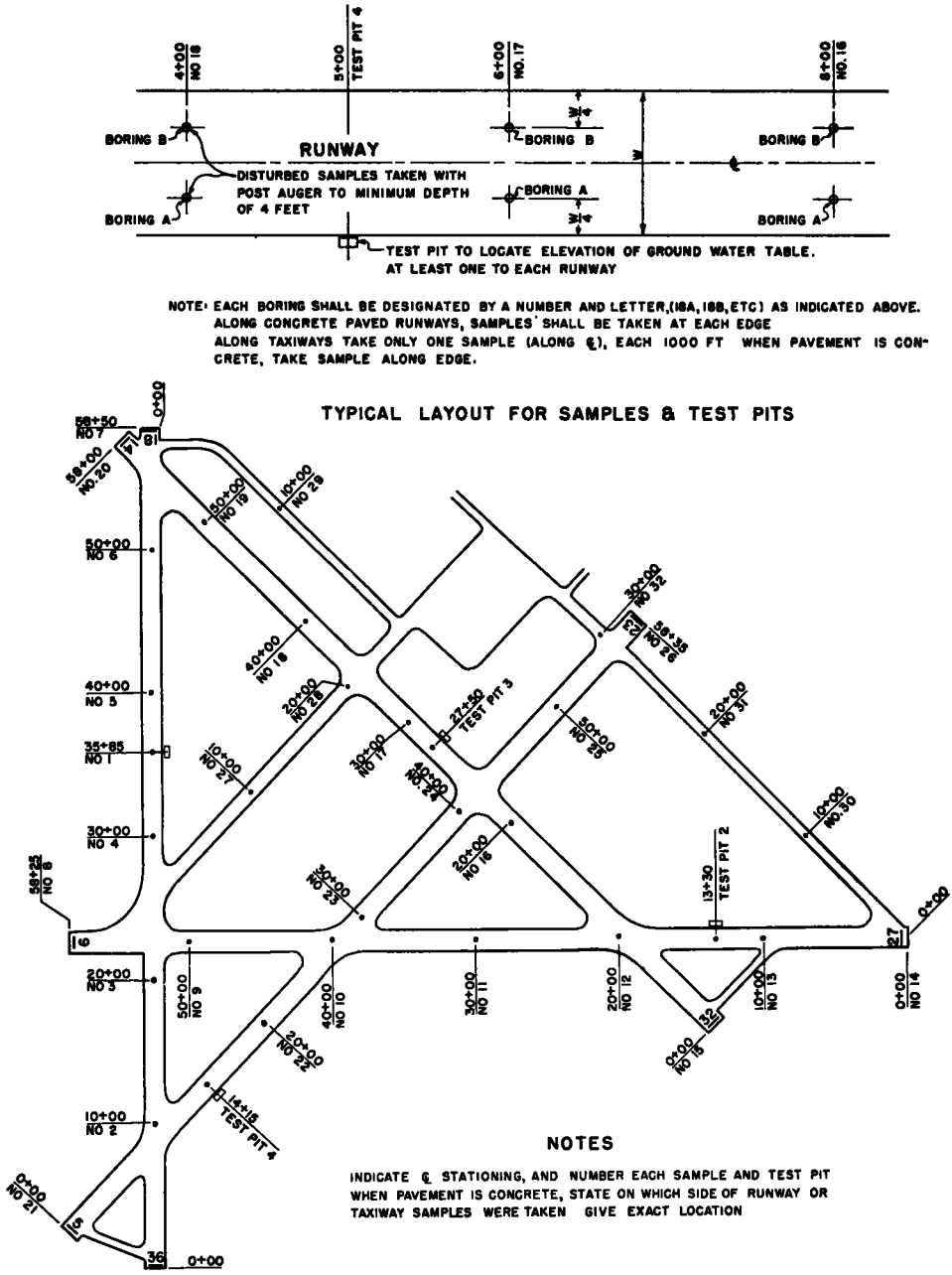


Figure 1. Typical Location Plan—Soil Borings

ture and grain size (sieve analysis) determinations were made in the mobile field testing laboratory. Asphalt, base course, and subbase samples taken from the 3- by 3-ft pavement section (Fig. 9) were tested in the laboratories of the Corps of Engineers, War Department. Undisturbed subgrade samples (Fig. 10) were taken in standard pipe sections, 2 ft in length with 4-in. inside diameter, which were swaged to a variable extent at the cutting

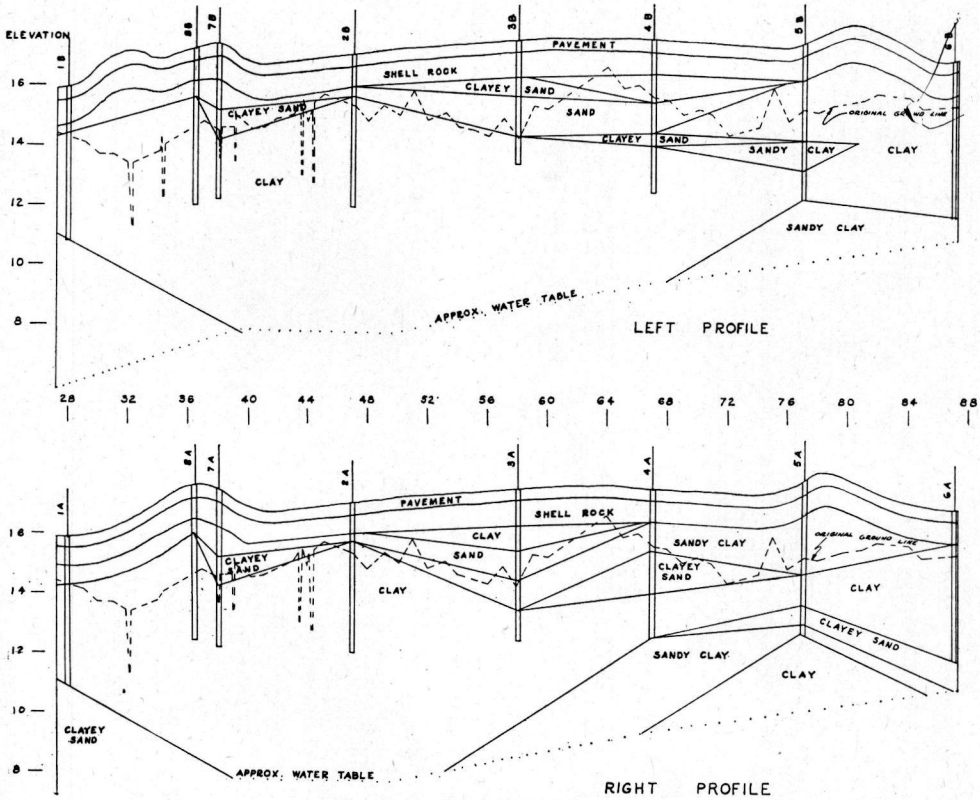


Figure 2. Typical Soil Profile

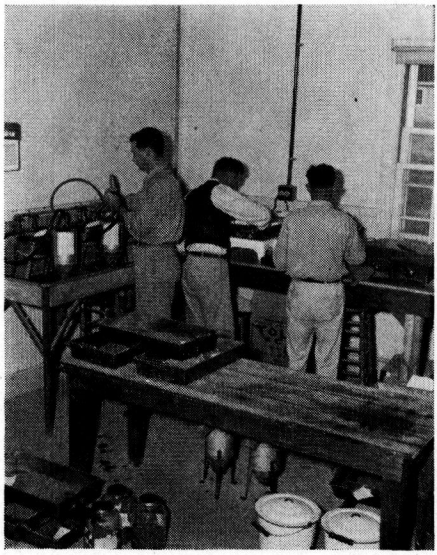


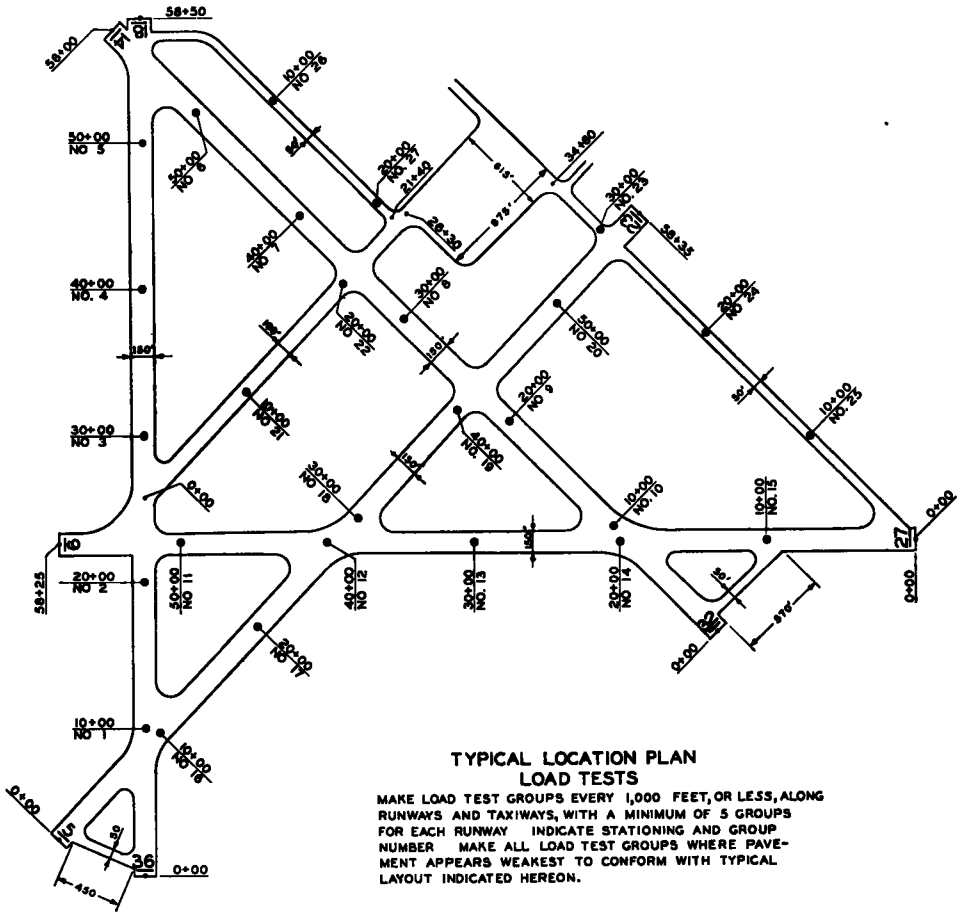
Figure 3. Mobile Field Testing Laboratory—NAS Pavement Evaluation Project

edge and suitable for obtaining full tube samples. The tubes with contents were capped securely with metal caps which were first taped and then sealed with paraffin. On arriving at the USED laboratories, the tube samples, four from each test location (Fig. 4), were each cut into two sections, one 9 in. long, representing the top 9 in. of subgrade, and one 15 in. long representing the lower 15 in. of the top 2 ft of subgrade.

Laboratory Tests

The laboratories of the Corps of Engineers made the following tests on the materials submitted.

Material	*Tests Performed*
Asphaltic pavement	a. total bitumen content b. aggregate gradation c. bulk specific gravity d. specific gravity of standard laboratory-prepared sample e. theoretical maximum specific gravity f. Hveem or Hubbard-Field stability of standard laboratory-prepared specimen g. Hveem or Hubbard-Field stability of field specimen h. swell test i. stripping test



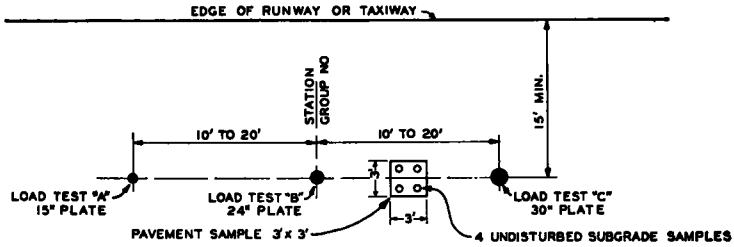


Figure 4. Typical Location Plan—Load Tests

Material	*Tests Performed*
Base and subbase course	a mechanical analyses b. modified AASHO compaction tests to obtain moisture-density curves c. liquid, plastic, and shrinkage limits on material passing the 40 (35 in many cases) mesh sieve
Subgrade	a. natural moisture content, density, and void ratio b. modified AASHO compaction tests c. mechanical analyses d. Atterberg limits e. triaxial compression tests (four) at constant minor principal stresses of 0, 5, 10, and 15 psi.

Where the subgrade had little or no cohesion, samples of disturbed soil were taken in lieu of undisturbed samples. In this case the laboratory omitted tests (a) and (e) in the foregoing list of subgrade tests and the data

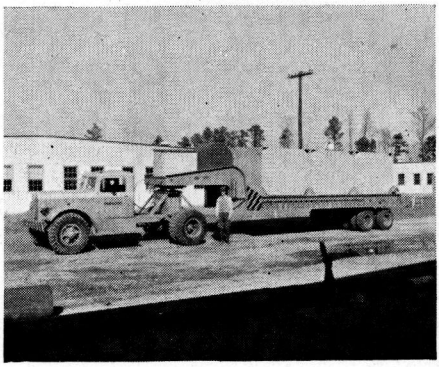


Figure 5. Loaded Trailer used for Reaction in Plate Loading Tests

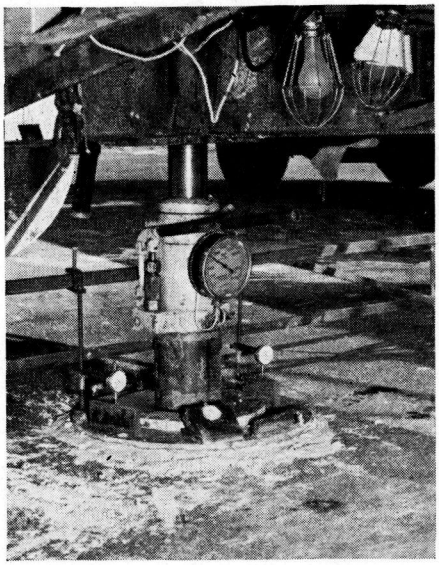


Figure 6. Loading Test on the Pavement with the 24-in. Plate

for (a) were obtained in the field. The four triaxial tests were made on the top 9 in. and also on the lower 15 in. of the undisturbed tube samples. Tests were carried to failure but the data of special interest were the deviator stresses corresponding to small strains.

In general, standard AASHO procedures were followed in the tests with asphalt and base course materials but in the case of asphalt surfacing, recent modifications of older pro-

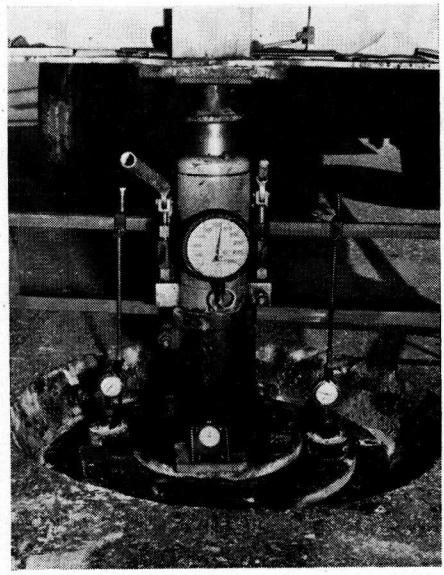


Figure 7. Loading Test on the Subgrade with the 30-in. Plate

Figure 8. Equipment for In-Place Density Measurements

cedures in light of research conducted by the Corps of Engineers and the Asphalt Institute, were utilized.

PRESENTATION OF DATA

An attempt is made at this time to indicate only general trends. A large mass of data have accumulated and a thorough analysis of them will require considerably more time than has been available. No final conclusions are warranted at present.

Figure 9. Stage 1 in Pavement Sampling—Asphalt Surfacing Removed

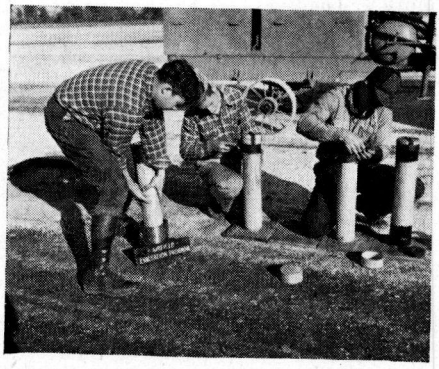


Figure 10. Capping and Sealing Tubes Containing Undisturbed Soil Samples

It is desired first to present very briefly an illustration of the procedure followed in evaluating the safe maximum wheel load bearing capacity of an individual runway at a typical Naval Air Station. In Figure 11, the unit load that produced 0.2 in. of settlement on a given 15-, 24- or 30-in. plate is plotted against the plate area. The numbers of the plotted points are the designations of the test locations in the paved area. An average curve is drawn through the points obtained by averaging the unit loads, for each of the three plate sizes, that corresponded to a 0.2-in. settlement. The average curve shows that in this case the unit load required to produce 0.2 in. of settlement decreased, on the average, as the plate area increased. The average curve should be more reliable in showing a general trend than a curve drawn from the data obtained from one test location. Next, a second curve, designated as the minimum is drawn. Two restrictions are imposed on the mimimum curve. First, it must be as nearly parallel as possible to the average curve but at the same time it must

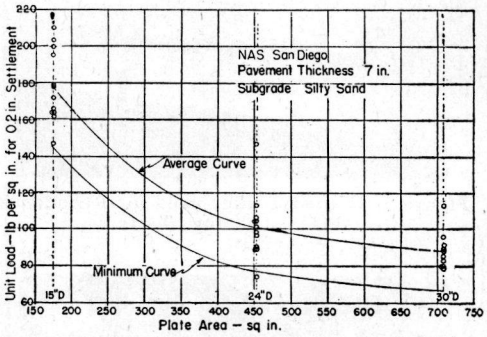


Figure 11. Graphical Procedure in Computing Safe Maximum Wheel Load Capacities

Tire Pressure *psi*	Contact Pressure *psi*	Safe Max. Wheel Load *lb*
60	66	46,500
70	77	34,500
80	88	32,000
90	99	31,000
100	110	30,000
110	121	28,500
120	132	27,500

pass through or near the minimum points. The minimum curve is then used for computing the wheel load bearing capacities tabulated in Figure 11 as follows. The actual contact pressure of tire on pavement is considered to be 110 percent of the tire pressure. If the tire pressure is 100 psi, the tire contact pressure is assumed to be 110 psi which, in Figure 11, corresponds to a tire footprint area, considered as circular, of 275 sq in. Then the total load is 275 x 110 = 30,250 lb, or 30,000 lb to the nearest 500 lb. There is no point to estimating the wheel load capacity more closely than to 500 lb since plane wheel loads cannot be estimated closely. In Figure 11, the plate area is taken as the tire contact or "footprint" area, assumed as circular, and the unit load on

the ordinate scale is used as tire contact pressure (110 percent of tire pressure).

By this procedure, a given asphalt surfaced flexible type pavement has wheel load bearing capacities that depend on tire pressures and the higher the tire pressure, the smaller is the wheel load for which the pavement is rated.

It is interesting to note that the pavement represented in Figure 11 has accommodated planes having wheel loads of from 20,000 to 40,000 lb, with tire pressures close to 85 psi or more without visible signs of rutting or failure. In many cases it is difficult to obtain complete information concerning the number of landings per day and the types of planes that used pavements both during the war and the year following the war. However, thus far in the studies there seems to be no evidence of pavement failure under wheel loads computed to be safe according to the foregoing procedure provided that the asphalt surfacing had sufficient toughness and thickness to resist scuffing and abrasion. It may develop in the future and in the light of additional and more pertinent data that the allowable deflection, 0.2 in. is too high for design and evaluation purposes. However, in the light of present information and experience, the Bureau of Yards and Docks considers this limit of deflection or settlement under plate loading as a reasonable index of pavement support. In 30 complete reports from 30 air stations there was no instance of surface cracking or other indication of failure at the site of plate loading tests where the deflection did not exceed 0.3 in. Deflections of from 0.3 to 0.5 in. did, however, in many instances induce cracking in the asphalt surfacing.

The effect of loading repetitions is not considered important when loading on used pavements that have already been repeatedly subjected to heavy traffic. However, in the construction and loading of trial pavement sections, in the design of new pavements, the published procedure of the Committee on Design of Flexible Type Pavements (*2*) which requires several repetitions of load application and load release has considerable merit.

Subgrade Moisture and Density Conditions.

The condition of subgrades under asphalt surfaced pavements was considered from the outset to be the most important variable factor requiring study in this investigation. Immediately preceding and during the war when there were few authentic data to indicate the degree of saturation and softness of the subgrade that could reasonably be expected subsequent to construction and use of an asphalt surfaced pavement, it was usually assumed that the subgrade would become quite wet and soft. Data obtained in the course of the Navy's pavement studies have not substantiated this belief. However, the studies do indicate quite variable subgrade conditions, affected or influenced by the type of subgrade, type of pavement, volume and kind of traffic, and climatic conditions.

The term "saturated subgrade" in this connection may mean next to nothing. A soil may be 100 percent saturated with almost any void ratio, ranging from 0.4 to 3.0. The density, moisture content, void ratio, and physical characteristics must all be stated to present a true and adequate picture of conditions. During war construction when time was of the essence, moisture and density control in subgrade preparation and compaction were usually not realized. Only under unusually favorable weather conditions was there any semblance of such control. Moreover, records of field control data that should have been obtained and kept during paving are conspicuously lacking. Consequently, it has been impossible in practically every instance to compare the subgrade conditions of a field used for two or more years with the conditions obtaining at the time of paving.

Although this report is primarily concerned with flexible type pavements, the subgrade data of Table 1, are for subgrade conditions under both asphalt surfaced and concrete pavements. A fair comparison of these conditions for the two different pavement types is hardly realizable, however, owing to the fact that subgrade preparation for concrete pavements at the air stations reported in Table 1 was usually considerably less emphasized as a factor of importance than obtained in the case of the flexible types of pavements, there was a general tendency to compensate for weak subgrades by designing concrete slabs of greater thickness. The instances where (by pumping action with silt and clay subgrades or by progressive densification of an initially loose sand subgrade under dynamic loading on the slab) concrete slabs were only partially supported at corners could no doubt have been prevented by

TABLE 1
SUBGRADE CONDITIONS

Field and Subgrade Soil Types	In-place Density	In-place Moisture	In-place Voids Ratio	In-place Saturation	Max. Compacted Density	Optimum Moisture	Plastic Limit
(A) Flexible Type Pavements							
	lb per cu ft	*% of dry wt*		*%*	*lb per cu ft*	*% of dry wt*	
So. Whiting, Pensacola, Fla. A-3 and A-2							
avg(24)[a]	109.7	6.7	0.522	34.1	121.7[b]	9.8	12.6
max	120.6	12.5	0.642	76.1	126.3	12.5	16.1
min	101.8	3.3	0.377	17.1	116.1	8.7	8.9
Barin Field, East Pensacola, Fla. A-2 and A-3							
avg(21)	105.5	5.2	0.588	24.8	115.4[b]	11.1	11.9
max	114.0	9.7	0.706	49.5	121.6	13.5	14.6
min	97.7	2.5	0.464	14.3	108 6	9.7	9.7
No. Whiting, Pensacola, Fla. A-2, A-3 and A-1							
avg(41)	107.4	6.7	0.555	33.4	N. D.[c]	N. D.	13.6
max	115.8	17.2	0.695	83.0			22.1
min.	97.7	2.6	0.439	11.3			N. P.[d]
Memphis, Tenn. A-4							
avg(21)	100.7	19.1	0.69	74.9	116.2	13.5	23.3
max	114.6	23.1	0.83	94.5	116.9	14.0	30.0
min	92.0	15.2	0.47	54.8	115.5	13.0	18.0
Jacksonville, Fla. A-3 and sand							
avg(21)	101.9	9.0	0.669	38.1	100.9	12.7	N. P.
max	111.3	15.6	0.776	68.5	104.3	15.5	
min.	93.7	2.8	0.511	11.3	97.5	9.7	
Oak Grove, N. C. A-3 and sand							
avg(12)	110.9	6.17	0.506	34.0	107.8	11.7	N. P.
max	116.0	11.1	0.625	66.7	113.2	13.8	
min	102.5	2.4	0.436	10.3	102.7	9.9	
Waldron, Corpus Christi, Texas ML[e] (silty sand, very fine sand) A-3 and A-4							
avg(13)	107.2	6.8	N. D.[c]	N. D	103.8	11.8	18.8
max	117.0	9.0			105.0	13.4	19.0
min	101.0	5.0			99.0	11.4	16.0
Main Station, Corpus Christi, Texas Sand and Silty Sand[e]							
avg(20)	94 3	7.2	N. D.	N D.	99.7	11.5	17.2
max	98.0	15.5			102.0	11.7	18.7
min	90.8	2.9			99.5	10.4	17.1
Rodd Field, Corpus Christi, Texas A-4, A-6 and A-7							
avg(14)	102 6	18.5	0.674	92.0	114.5	15.4	21.7
max	114.0	23.0	0.832	97.6	120.5	16.8	22.0
min	91.0	14.0	0.623	81.7	112.1	12.3	21.0
Chase Field, Corpus Christi, Texas A-4, A-6 and A-7							
avg(13)	114.7	8.3	0.555	63 8	124.5	10.1	16.1
max	128.5	13.2	0.770	82 4	126.5	12.2	20.0
min	106.0	4.6	0.433	32.6	121.0	8.6	16.0
Cuddihy Field, Corpus Christi, Texas A-4 and A-7							
avg(22)	100.8	21.5	0 838	94.5	110.4	16.9	24.0
max	114.0	29.6	1.137	99.6	117.6	18.7	24.6
min	91.2	6.9	0.560	88.8	107.0	14.2	20.0
North Field, Kingsville, Texas A-2, A-4 and A-7							
avg (16)	101.4	12.1	0.714	61.6	117.6	12.5	17.8
max	112.0	18.0	0.865	80.9	123.0	15.5	20.0
min	94.0	7.0	0.501	44.3	112.0	10.5	16.0
Cabaniss Field, Corpus Christi, Texas A-4, A-6 and A-7							
avg(14)	102.9	26.0	0.630	93.1	113.4	15.4	21.2
max	115.0	34.4	0.888	98.3	117.2	18.7	29.0
min.	91.0	17.2	0.391	79.1	107.8	13.4	19.1

TABLE 1—*Continued*

Field and Subgrade Soil Types	In-place Density	In-place Moisture	In-place Voids Ratio	In-place Saturation	Max. Compacted Density	Optimum Moisture	Plastic Limit
(A) Flexible Type Pavements—*Continued*							
	lb per cu ft	*% of dry wt*		*%*	*lb per cu ft*	*% of dry wt*	
South Field, Kingsville, Texas A-4, A-6 and A-7							
avg(14)	99.5	14.2	0.756	66 6	112.2	16.1	19.7
max	109.0	22.9	0.902	92.9	117.0	18.4	28.0
min	90.0	10.9	0.588	44.2	109.6	13.8	24.4
Clinton, Okla. A-4, A-6 and A-7							
avg(26)	111 4	15.4	0.510	81.9			16.5
max	118.1	18.5	0.623	95.7	N. D.[c]	N D.	24.2
min	104.0	10.7	0.420	51 6			16.0
Seattle, Washington, Sandy Point NAS A-3, A-4 and A-7							
avg(18)	116.8	14.2	0.465	69.5	125.9	11.0	17.1
max	134.9	25.2	0.810	88.2	139.5	21.2	33.0
min	86 5	7.6	0.348	30.7	90.4	6 7	N P
NAS Livermore, Calif. A-2 and A-4							
avg(28)	116.6	12.3	0.451	74.9	128.7	9.0	15.8
max	129.0	17.9	0.587	87.4	140.0	12.0	18.3
min	104.1	4.3	0.314	34.3	120.6	7.0	14.0
Edenton, N. C. A-2, A-3, A-4 and A-7							
avg(22)	113.8	15.5	0.471	88.5	119.2[b]	12.8	16.5
max	122.0	21.3	0.625	98.0	123.4	17.0	17.8
min	103 2	10.5	0.370	69.0	110.9	11.0	15.0
(B) Concrete Pavements							
NAS Crows Landing, Calif.[f] A-2, A-4 and A-7							
avg(17)	100.7	21.0	0.651	86 6	114.1	14.2	20.2
max	108.6	24.0	0.792	100.0	121.5	17.0	27.0
min	92.0	18.4	0.522	61.4	106.8	11.5	16.7
Glenview, Ill.[f] A-2, A-4 and A-6							
avg(13)	90.8	21.1	0.662	76.8	114 7	13.9	24.4
max	114.5	29.9	0.927	94.8	126.5	17.4	28.3
min	84.9	16.9	0.498	72 1	106.3	10.0	19.6
Seattle, Washington A-3, A-4 and A-7							
avg(7)	105.3	17 0	0.639	71.9	124.8	11.3	16.7
max	118.2	33.0	0.970	91.5	135.0	14.5	19.2
min	85.2	11.2	0.450	49.2	109.0	8.5	15.5
NAS, Livermore, Calif. A-2 and A-4							
avg(25)	111 7	12.6	0.513	68.5	128 4	8.3	15.0
max	116 2	13.8	0.575	82.8	129 0	8.3	15 1
min	107.2	11.5	0.450	54.1	127.8	8.2	15.0
Jacksonville, Fla A-3 and Sand							
avg(6)	103.1	10.2	0.614	44.7	104.8	12.5	
max	105.6	19.8	0.664	88.3	119.2	13.7	N. P.[d]
min	99.7	5.8	0.580	24.7	99.2	10.4	
Memphis, Tenn A-4							
avg(8)	98.8	23.2	0.700	89.3	116.4	13.6	23.1
max	109.8	26.8	0.881	98.3	119.6	14.0	26.0
min	87.5	18.4	0.510	74.1	115 5	12.4	21.0

[a] Figures in parentheses refer to the number of tests averaged
[b] Compaction according to the standard AASHO procedure
[c] Not determined
[d] Non plastic
[e] Notation as given in USED report
[f] Entire field paved with concrete.

more adequate subgrade preparation before paving. Here again, speed, being the most important consideration during war-time construction, it was inevitable that certain very

desirable features of construction, such as the one here considered, be neglected to a considerable extent.

In the case of concrete pavements the moisture content of subgrades may have tended to be generally higher than that under asphalt surfaced pavements at the time of paving, and it may be that this difference persisted after construction. Faulty joint seals and poor maintenance contributed further to wet subgrades under concrete pavements.

It should also be mentioned that test locations on flexible pavements were selected in general on the basis of subgrade soil surveys, areas where the surfacing had failed badly were not loaded if such areas were not representative of the taxiway, runway, etc., as a whole. Loading on pavements were often made, however, in "bird baths" and "duck ponds" or where checking and cracking of the surfacing were very noticeable, when this was a more or less prevalent condition.

TABLE 2

Air Station	Subgrade Soil Types	In-place Moisture / Optimum Moisture
		%
So Whiting, Pensacola, Fla	A-2, A-3	53
Barn Field, Pensacola, Fla.	A-2, A-3	43.8
Jacksonville, Fla.	A-3	71.0
Oak Grove, N. C.	A-3	52.7
Waldron Field (Tex.)	ML, A-3, A-4	57.5
Main Station (Corpus Christi)	sand and silty sand	62.5
Chase Field (Tex.)	A-4, A-6, A-7	82.0
No. Field, Kingsville (Tex)	A-2, A-4, A-7	96.7
So. Field, Kingsville (Tex).	A-4, A-6, A-7	88.1

Table 1 is arranged in two parts; (A) flexible type pavements, 18 fields, and (B) concrete pavements, 6 fields. This represents a total of 22 fields of flexible pavements, since two fields of Table 1(B), Crows Landing and Glenview, are paved entirely with concrete.

In a study of Table 1, it may be noted first that of the 18 stations listed under Table 1(A) the average in-place subgrade moisture content (top 9 in. of subgrade) of nine of the fields, was actually less than the measured laboratory optimum moisture. These fields and the soil types are shown in Table 2. The same condition obtained for one of the six air stations reported in Table 1(B) for concrete pavements, that is, the NAS field at Jacksonville, Fla., which has a sand subgrade.

On the nine fields listed in Table 2 the subgrade was predominantly sand for the first six and predominantly fine grained soils, silt, and clay, for the remaining three. The surfacing of all nine of these fields was at least in fair condition at the time of sampling. The asphalt surfacing at Rodd, Cuddihy, and Cabaniss Fields, Corpus Christi, Texas were so badly torn and abraded as to preclude the use of certain runways, taxiways, and landing mats at the time of subgrade sampling. In these three cases, samples were taken both where the surfacing had failed badly and where failure was less noticeable, both conditions being well represented. It is true that the subgrade conditions for these three fields were not representative of normal subgrade conditions under good asphalt pavements.

It should be mentioned in this connection that with the exceptions noted in Table 1, the optimum moisture percentages reported were those obtained by the modified AASHO compaction procedure, used by the Corps of Engineers, which yields somewhat higher densities and lower optimum moistures than those obtained in standard highway practice.

It is well also to compare the in-place moisture contents with the plastic limits. Note that for the concrete pavements (Table 1(B)), the average in-place moisture content was approximately equal to the plastic limit and that only in one instance out of 76 (maximum value for NAS, Seattle) did the in-place moisture content greatly exceed the plastic limit. Such an observation fails to substantiate the older and more pessimistic view that subgrades under pavements become very wet and soft.

Excluding Rodd, Cuddihy and Cabaniss fields in Table 1(A), one individual subgrade sample, namely the maximum in-place moisture percentage of 21.3 for Edenton Field, exceeded the maximum plastic limit, 17.8, considering only the fields for which plastic limit values are given. That is, of a total of 289 subgrade soil samples taken from under asphalt pavements in reasonably good condition, one was found which had an in-place moisture content greater than the maximum observed plastic limit for the field and seven were found whose in-place moisture contents exceeded the average plastic limit of the subgrade samples of the field. Neither do these

simple facts, observed in the field and laboratory, incline one to assume extreme conditions of subgrade moisture as a basis for airport pavement design.

The percentage saturation (Col-6, Table 1) in itself means nothing with reference to subgrade stability. A soil may be 100 percent saturated, having its voids completely filled with water, and yet its void ratio may be 0.3 or 3.0. Capillary saturation of clays and silts is frequently observed and actually increases the cohesion of such soils. Thus if a soil sample, saturated by capillarity, is located at 10 ft above ground water, it is under $10 \times 62.4 =$ 624 lb per sq ft capillary pressure in place. This is a uniform "all around" pressure and appears as cohesion in tests. If the water table rises above the sample, the apparent cohesion caused by the 624 lb per sq ft capillary pressure is lost. The sample is saturated both before and after such loss of cohesion.

However, it is interesting to observe only one instance of 100 percent saturation (NAS, Crows Landing, Calif.) although there were several instances where the percentage of voids filled with water exceeded 90. Most of the saturation in place was observed as that due to capillarity. This type of saturation differs considerably from that produced when a lightly loaded compacted sample is submerged in water for four or more days in a cylindrical mold, open at the ends, as in the CBR test. This test simulates subgrade conditions when a free water surface exists under the pavement and all apparent and most of the true cohesion is lost. Such a condition was never observed in the course of this investigation of pavements at Naval Air Stations where the asphalt surfacing was suitable for traffic. Thus observations indicate strongly that subgrades usually become soft and wet because the surfacing is torn rather than that the surfacing fails because the subgrade is wet and soft. Failure of the surfacing has been observed to occur through overloading with the subgrade moisture not far from optimum. Subsequent wetting and softening of the subgrade soon followed. Cause and effect are easily and needlessly confused in this connection.

On the fields listed in Table 1(A) and (B), the water table was not less than 4 ft below the pavement in any instance. Other fields for which reports are not yet complete are known to have ground water within 2 ft of the pavement. Such unfavorable ground water conditions are of course reflected in the Navy's trial section procedure for designing thicknesses of flexible type pavements (*3*).

A recapitulation of density relations in Table 1 is given in Table 3. In Table 3, the ratio of the average in-place density to the average laboratory compacted maximum density, expressed as percent, is given in the second column. The third column shows the ratio of the maximum in-place density to the maximum compacted density and the fourth column shows the ratio for the corresponding minimum density values. Values are given to the nearest whole number.

TABLE 3
RATIO OF IN-PLACE DENSITY TO MAXIMUM DENSITY

Field	In place density ÷ maximum density × 100		
	Averages	Maximums	Minimums
(A) Flexible Type Pavements			
	%	%	%
So. Whiting, Fla.	90	95	88
Barin, Fla	91	94	90
Memphis, Tenn	87	99	80
Jacksonville, Fla	101	108	96
Oak Grove, N. C.	103	102	100
Waldron, Tex.	104	111	102
Main Station, Corpus Christi, Tex.	95	96	91
Rodd, Tex	90	95	81
Chase, Tex	92	102	88
Cuddihy, Tex	91	97	85
No. Kingsville, Tex.	86	91	84
Cabaniss, Tex	91	98	84
So Kingsville, Tex.	89	93	82
Seattle, Wash.	93	97	96
Livermore, Calif.	91	92	86
Edenton, N. C.	95	99	93
(B) Concrete Pavements			
Crow's Landing	88	89	86
Glenview, Ill.	79	91	80
Seattle, Wash	84	88	78
Livermore, Calif.	87	90	84
Jacksonville, Fla.	98	89	101
Memphis, Tenn.	85	92	76

The following facts are apparent from Table 3:

1. The in-place densities of the sandy subgrades under flexible pavements at three fields, Jacksonville, Oak Grove and Waldron, exceeded the maximum densities obtained by compaction in the laboratory by the modified AASHO procedure. Under a concrete pavement (Table 3(B)), Jacksonville, the same tendency for rela-

tively higher density in-place for sand than for other soil types is observed.

2. For flexible type pavements at 16 fields (Table 2(A)), the lowest density in-place observed was 81 percent of maximum. This was at Rodd Field, where asphalt surface failure was pronounced and extensive.
3. For concrete pavements at six fields, the lowest observed in-place density was 76 percent of maximum. This individual case was in an area where the joint seals had failed very noticeably. Joint seal failure was also observed at Glenview where the average in-place density was 79 percent of maximum.

The writer considers that the observed densities of the top 9 in. of in-place subgrade soils were generally considerably higher than those which would be characteristic of soils in the California bearing ratio test, used as a basis for design (*4*).

A comparison of subgrade conditions under concrete and flexible pavements is considerably limited in this report by the relatively fewer data available at this time for concrete pavements. On the basis of data herein reported, it does not seem that outstanding differences exist. Furthermore, it is repeated and emphasized that any fair and direct comparison in this connection is hardly possible owing to the fact that at the fields reported herein and at other fields included in this study and which will be included in a future report, it is obvious that there was generally more careful subgrade preparation for the asphalt surfaced than for the concrete pavements.

This writer also ventures the opinion, based on Table 3(B), that the procedure of reducing the modulus of subgrade reaction, k (obtained by loading with a 30-in. plate on the subgrade or base course) by making laboratory consolidation tests of soil from the site (*5*) may not be warranted.

In a discussion of a paper by Tschebotarioff (*6*) this writer mentioned the possible effect of vibration produced by plane wheel loads in gradually building up high densities of sand subgrades under pavements. The data more recently obtained tend to further support this supposition. Since traffic tends to be distributed on runways, this gradual and general increase of subgrade sand density tends to increase the bearing value of the pavement beyond that which it had when newly constructed. In taxiways there is some possibility of tracking which could produce troughs due to frequent load repetitions in the same area and consequent densification of subgrade sand in that area.

Factors Influencing Subgrade Moisture.

The main factors influencing the subgrade moisture and density conditions reported in Table 1 would be expected to be climatic conditions, drainage conditions, and pavement conditions. Table 4 gives a summarized description of these conditions at the fields at the time of sampling. It is apparent that a wide range of conditions was represented in this study of subgrade moisture.

Loading Test Data

Many extremely variable factors affect the stress-strain relationships as determined by plate loading on the pavement surface. Pavement properties and composition are likely as important as pavement thickness. Pavement properties depend on construction procedures, weather conditions during construction and other variable factors as well as on materials. Methods of pavement design or pavement evaluation based solely on laboratory tests fail to take into account two simple facts: (1) field compaction is very different from compaction of the same materials in the laboratory; and (2) the load spreading effect of the different layered components of a pavement depend not altogether on the composition and degree of compaction etc., of each separate layer but also upon the relative influence of each separate layer upon the others comprising the pavement. The variable factors are interrelated and this interrelationship can be reflected only in a test of the finished product, the pavement. As stated by D. P. Krynine, (*7*) "pavement building is a case of mass production and in all such cases of mass production much more evidence is furnished by the production itself than by mathematical analyses or laboratory experiments." The plate loading tests with all their limitations do constitute tests on the finished product. The pavement supporting value has not been predictable by attempted syntheses of the whole complicated structure from laboratory tests on the separate materials.

TABLE 4
SURVEY OF CONDITIONS AFFECTING SUBGRADE

Field	Pavement Description	Drainage Features	Climatic Conditions Immediately Preceding and During Subgrade Sampling
So. Whiting, Pensacola, Fla.	6 in. of mixed-in-place sand asphalt—Poor curing of asphalt	Poor—Practically no surface drainage—Water ponded at intersections—Transverse slopes practically zero	Apr and May of 1945—Wet and humid weather conditions
Barin Field, Pensacola, Fla.	Conditions about the same as at So. Whiting Field	Conditions same as at So. Whiting Field	Late summer and fall of 1945—Hot, humid and wet
No. Whiting, Pensacola, Fla.	6 in. of mixed-in-place sand asphalt—Asphalt well cured—Good design of mix	Good—Transverse slopes generally close to 1 percent—Sloping shoulders and swales for surface run-off are adequate—No ponding—Low ground water	Feb and Mar, 1945—Cool and comparatively dry
NAS, Memphis, Tenn.	Surface 3 to 5 in. sand-MCl asphalt—Base, 8 in. poorly graded gravel	Poor—Standing water on runways during sampling—Extensive surface cracks in asphalt surface—Flat transverse grades—Many "bird baths"—One runway without drainage ditches	Jan and Feb, 1946—A very wet season, prolonged and heavy rainfall—No freezing of subgrade
NAS, Jacksonville, Fla.	Surface, ¾ to 1½-in. dense A C—Base, 6 to 8 in. crushed limerock—Sub-base, none—Surfacing too thin, same failure	Good—Transverse grades ½ to 1 percent, grassed slopes, 20 on 1 carry surface water to St. Johns River—No sub-surface drainage structures—Ground water 4 ft or more below pavement	5¾ in. of rain from Jan 1 to Mar, 1946—Sampling done in Mar.
MCAS, Oak Grove, N. C.	Surface, 2 to 3 in. sand asphalt—bas, avg thickness 6 in. of crushed, stone—Sub-base none	Field drains by open ditch system into Trent River 1½ percent transverse runway slopes—Drainage is adequate	Sampling from Jan 20 to Mar 1, 1946—Cool and wet, no heavy rains but prolonged drizzles—No frozen ground
Waldron Field, Corpus Christi, Texas	Surface, 1 to 1½ in. sand asphal—base, 6 in (avg) compacted shell—sub-base, none	Very good—Good surface drainage Transverse slopes of 1 per cent or more; grassed sloping shoulders, swales	Sept, 1944—Heavy rains preceding and during sampling
NATB, Main Station, Corpus Christi, Texas	½ to 1½ in. hot mix and penetration surfacing—base 3½ to 5 in. of cold travel plant mix, sand, shell and emulsified asphalt—no sub-base	Conditions same as at Waldron Field	Conditions same as at Waldron Field, Sept, 1944
Rodd Field, Corpus Christi, Texas	Surface, thin penetration cover—base 5 in. of sand mixed with shell and emulsion—Sub-base, 5 in. sand with little clay	Extensive landing mats, flat and poorly drained, much standing water—Legs of mat fairly well drained—Surface failure extensive	Oct, 1944—Heavy rains preceded sampling—Clear, warm weather during sampling
Chase Field, Corpus Christi, Texas	Surface, 1 in rock asphalt—Base, 8 in. compacted Caliche—sub-base, 6 in. sand with little clay or loam	Well drained—No standing water in triangles between runways and very little on runways—Transverse grades about 1 percent—Surface drainage relied on	Oct 1944—Conditions about the same as those at Rodd Field
Cuddihy Field, Corpus Christi, Texas	1 in. plant mix, sand-shell-emulsion surface—5 in cold mix of shell, sand and emulsion for base course—No sub-base	Description about the same as that for Rodd Field	Oct. 1944—Conditions about the same as those at Rodd Field
North Field, Kingsville, Texas	1 to 1½ in. of rock asphalt surface—base 6 in of compacted Caliche on runways, 8 in. on taxiways—No sub-base	Generally good—Transverse slopes of ½ to 1¼ percent—Very little standing water—No ponding in triangles between runways	Sept, 1944—Heavy and frequent rains during period of sampling
Cabaniss Field, Corpus Christi, Texas	Thin emulsion, penetration surface—Base courses of 2 types, 5 in. shell, sand & emulsion and 6 in soil-cement—No sub-base	Soil-cement pavement in good condition—Elsewhere, surfacing had failed extensively—Drainage poor, similar to conditions at Rodd Field	Oct, 1944—Conditions about the same as at Rodd Field
So. Field, Kingsville, Texas	Same as No Field, Kingsville, Texas	Same as No Field, Kingsville, Texas	Same as No Field, Kingsville, Texas
NAS, Clinton, Okla.	Surface, 1½ in. densely graded gravel, hot-mix A.C—Base, poorly graded with much clay, 6 to 8 in. thick—Sub-base, none	Fair—Crowned runways, ½ to 1 percent, transverse slopes, side ditches sloping shoulders—No sub-surface drainage—Surface failure extensive	Mar, 1945—Wet conditions after thawing of frozen ground—Considerable snow and rain during Jan and Feb preceding sampling

TABLE 4—*Continued*

Field	Pavement Description	Drainage Features	Climatic Conditions Immediately Preceding and During Subgrade Sampling
NAS, Seattle, Washington	Surface, 4.7 in avg thickness, A C., laid in 2 courses—Base 6 in., avg bank run gravel, 40% passing no 200 sieve—Sub-base, none	Runways surrounded by french drains with 6 in. conductor drain tile, concrete catch basins, 200 ft O.C. collect water from drains—No sub-surface drainage—Transverse grade of runways ½ per cent	Jan, 1946—Sampling was done during the winter rainy season characterististic of this locality
NAS, Livermore, Calif.	Surface, 1 to 2½ in. hot plant-mix A.C.—Base, crusher run bank gravel 6 to 13 in —No sub-base	Runways, 1½ percent and landing mats 1 percent transverse slopes—Open ditches carry surface water to Western Pacific R. R. drainage system—Unpaved areas drain under taxiways via culverts	Jan, 1946—During Dec., 1945 preceding sampling, 15 in. of rainfall were recorded at this field
NAS, Edenton, N. C.	Surface, 2½ to 4 in. hot plant mix course sand-asphalt in 2 courses—Base, avg of 7 in crushed rock—Sub-base, mechanically stabilized sand and clay of quite variable thickness	A system of swales carries all surface water to swamps north of field, 1½ percent transverse grades on runways—Crushed stone drains lead from pavement to swales, these at 200 ft O.C.—Shoulders sometimes very wet	Apr, 1946—Moderate rains during the sampling period
NAS, Crows Landing, Calif.	Surface, unreinforced concrete, 6 to 9 in. at slab interiors—Base course, 12 to 24 in. of well graded pit-run gravel—Concrete in excellent condition	Fair. Asphalt paved shoulders—Runways and taxiways not crowned but have 1 percent transverse grades in one direction—Gravel drains lead from edges of pavement to open ditches—These drains at 150 ft O.C	Mar, 1946—No appreciable rainfall during period of sampling—Moderate rains preceding this period
NAS, Glenview, Calif.	Surface, unreinforced concrete —Base course, none—Some joints require resealing—Slab thickness 6 to 9 in. at interiors	No authentic information available —Apparently there is no sub-surface drainage—Transverse grades not known but appear to be very small—Concrete good—Joints poor in some areas	Oct and Nov, 1945—Moderate rains with conditions unfavorable for drying

TABLE 5

EFFECT OF ASPHALT SURFACING THICKNESS ON PAVEMENT SUPPORTING VALUE—DATA FROM MAIN STATION, CORPUS CHRISTI, TEXAS

Pavement	Number of Tests Averaged	Avg. Unit Load to Produce 0.2-in. Settlement		
		15-in plate	24-in. plate	30-in. plate
		psi	*psi*	*psi*
½- to ½-in. asphalt penetration surface Avg. of 4½ in. of bituminous sand and shell base course	10	109	63	50
Surfacing 1.8-in of hot-mix A.C.—Base course same as foregoing	8	126	79	63

To evaluate accurately the effect or importance of any one variable, for example the thickness of a specific type of asphalt surfacing, all other variables would need to be known and controlled. It is likely that such control could only be realized in pavement sections constructed solely for experimental purposes and even then the most ideal conditions would be required. Further, such experimental sections must necessarily be subjected to normal traffic or simulated normal traffic conditions and not to accelerated tests that have no true resemblance to normal traffic distribution.

In the analysis of data obtained in this particular study, the effect or influence of any single variable factor has been obscured in nearly all instances by variations in the several other factors. Nevertheless, the influence of a single factor often is so outstanding that it may show a strong general trend despite a high degree of variability of the other factors. As an example consider the data of Table 5, wherein is shown the effect of varying the thickness of asphalt surfacing on the unit load on the three plate sizes required to produce a 0.2-in. deflection.

On the basis of Table 5, an increase of about 1.5 in. in surface thickness, all other variable conditions being approximately the same, resulted in an increase of about 20 percent in pavement bearing value. This instance is typical. Others may be presented when all of the data are received and analyzed.

Table 6 shows, for two fields with silt-clay subgrades, the indicated effects of total pavement thickness, modulus of compression, C, of the subgrade, and the relative density of the

base course on the bearing value of the pavement. The modulus, C, was determined according to the procedure described in *Public Roads* (*8*) and is a laboratory determined value.

The data of Table 6 are also typical. A complete statistical study of all the fields will require a great deal of time but it is hoped that this may be accomplished and presented in a later report. The four variable factors of the first column of Table 6 varied independently, each of the others. The relative importance of each one is not indicated by these data. Yet each of the four was apparently sufficiently important to effect small differences in

TABLE 6

AVERAGE RESULTS, NORTH AND SOUTH FIELDS, KINGSVILLE, TEXAS, SHOWING INFLUENCE OF VARIABLE FACTORS.—(A) ABOVE AVERAGE VALUE, (B) BELOW AVERAGE VALUE

Variable Factor	Average Value of Variable	Average Unit Load Required to produce 0.2-in. settlement					
		15-in. plate		24-in. plate		30-in. plate	
		(A)	(B)	(A)	(B)	(A)	(B)
		psi	*psi*	*psi*	*psi*	*psi*	*psi*
Total pavement thickness	7 in.	134	113	76	70	68	55
Modulus C, of subgrade	3150 psi	132	115	75	69	68	54
Relative density of subgrade	87.5 percent of maximum	128	120	75	72	65	58
Relative density of base course	94.5 percent of maximum	133	116	76	70	68	55

bearing value that could not be completely obscured or offset by the fact that the other factors varied in a manner to oppose this effect. It should be noted however that two of the variables, modulus, C, and density of subgrade, varied simultaneously; as one increased the other increased correspondingly. The relative values of these two properties are not apparent from these data although it is apparent that measurements of either subgrade modulus or density yield data that are indicative of the subgrade bearing value which in turn affects the pavement bearing value if all other variables are controlled within certain limits.

The modulus, C, can be also computed from the expression derivable from the point load formula of Boussinesq (*8*).

$$C = \frac{3pa}{2S} \quad \ldots\ldots\ldots\ldots (1)$$

In this expression,

C = modulus of compression of the subgrade
p = unit load, uniformly distributed
a = radius of circular loaded area
S = settlement

In Figure 12, the average C obtained by loading on the subgrade at six of the eight fields at Corpus Christi, Texas (all of the eight fields except Rodd and Cabaniss) is plotted against the average (for the field) unit

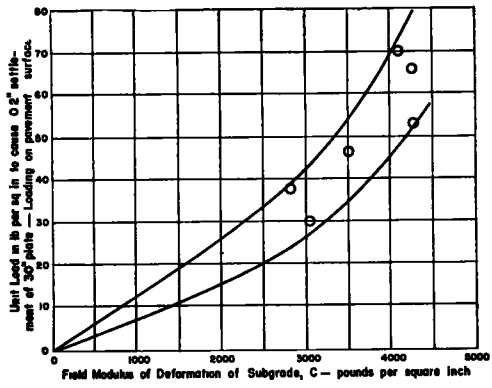


Figure 12. Relation Between Field Modulus of Deformation, C, of Subgrade, and Unit Load on Pavement Required to Produce 0.2 in. of Settlement—Average Data for 6 Fields with Different Flexible Pavements

load on the 30-in. plate, loading on the pavement, to produce 0.2-in. settlement. In general, the field value of C, so computed, was not in good agreement with the laboratory C most likely owing to the fact that in field loading, a depth of six ft or more of subgrade influences the test whereas the averaging of C values for subgrade samples within the 2 ft of ground under the pavement is somewhat like averaging the number of horns per animal from a given number of horses and cows. One is an effective average and the other is simply a numerical one. From a practical standpoint the field tests seem to eliminate many questionable assumptions. The C (or E as generally written) was never specified by Boussinesq or anyone else, apparently, as being necessarily a laboratory determined value.

The two curves of Figure 12 are arbitrary and are drawn simply to encompass the group of plotted points and show the range of divergence with rather widely different subgrade conditions but with comparatively less variations in pavements.

Figures 13 and 14 show similar data except that in these two cases individual values rather than averages are plotted. In these two

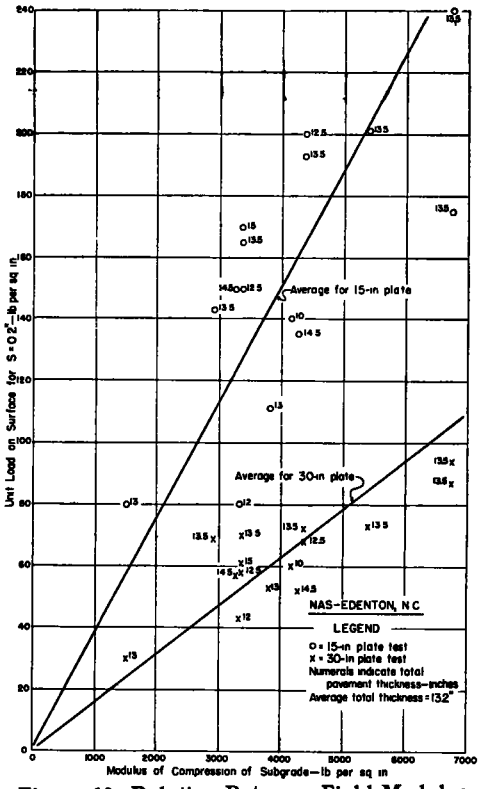


Figure 13. Relation Between Field Modulus, *C*, of Subgrade and Unit Load on Pavement Required to Produce 0.2 in. of Settlement

figures the unit loads are plotted on the ordinate scale and total pavement thicknesses are indicated for each plotted point. It is noted that the divergence of points is greater for the 15- than for the 30-in. diameter plate, a condition to be expected since, all other things being the same, the relative importance of the bearing value of the subgrade should increase as the loaded area increases. Considering an extreme case, the load on a small coin, such as a dime, in surface loading to

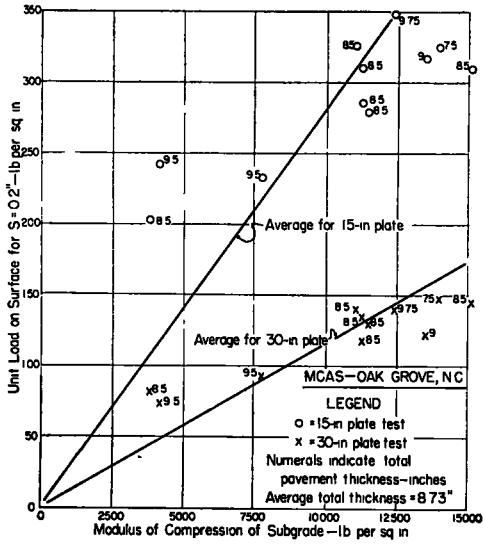


Figure 14. Relation Between Field Modulus, *C*, of Subgrade and Unit Load on Pavement Required to Produce 0.2 in. of Settlement

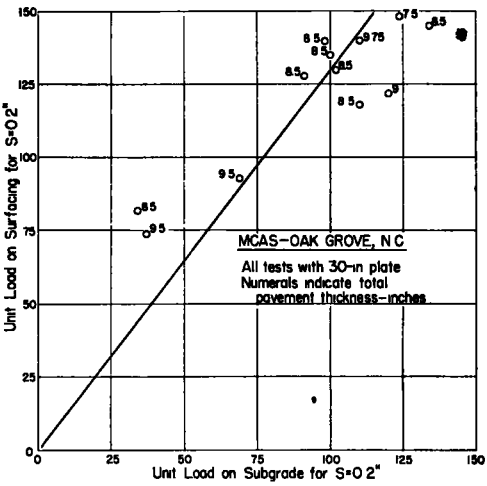


Figure 15. Relation Between Unit Loads on Subgrade and Pavement Surface

	Avg	*Max*	*Min*
Thickness—*in.*	2 6	3 0	2 5
Base Course			
Thickness—*in.*	6 2	7 0	5 0
Density—*lb per cu ft*	124 9	142 0	111 0
Moisture—*percent*	5 0	5 9	4 0
Subgrade			
Density—*lb per cu ft*	110 9	116 0	103 0
Moisture—*percent*	6 2	11 1	2 4

effect a given settlement, would be scarcely affected or influenced at all by the subgrade.

The unit load on pavement for 0.2-in. settlement is plotted against the unit load on the subgrade for the same settlement, using the 30-in. plate in both cases, for the Oak Grove, Livermore, Edenton and Seattle Naval

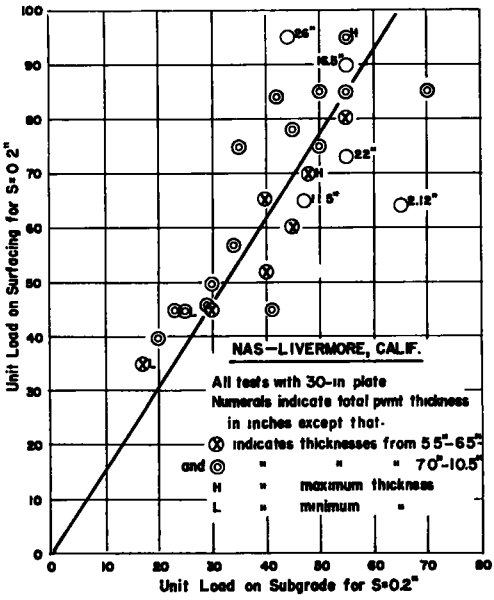


Figure 16. Relation Between Unit Loads on Subgrade and Pavement

	Avg	Max	Min
Surface			
Thickness—*in.*	1 64	6 50	1 00
Base Course			
Thickness—*in.*	7 52	24 0	1.0
Density—*lb per cu ft*	139 3	145 5	131.0
Moisture—*percent*	4 7	6.4	2.4
Subgrade			
Density—*lb per cu ft*	116 6	129 0	104 1
Moisture—*percent*	12 3	17.9	4 3

and Marine Corps Air Stations in Figures 15, 16, 17 and 18. Other pertinent data are shown in these figures. The slopes of the straight lines (averages) in the four figures are 1.29 1.54, 1.76 and 1.62 respectively. The slopes indicate the probable range in the basic relationship between subgrade and pavement bearing values under rather widely different conditions for both pavements and subgrades. For a ratio or slope of unity, the subgrade, on the average, would be equal in supporting value to the pavement on the subgrade, granting that the allowable deflection of 0.2 in. is a good measure of supporting value.

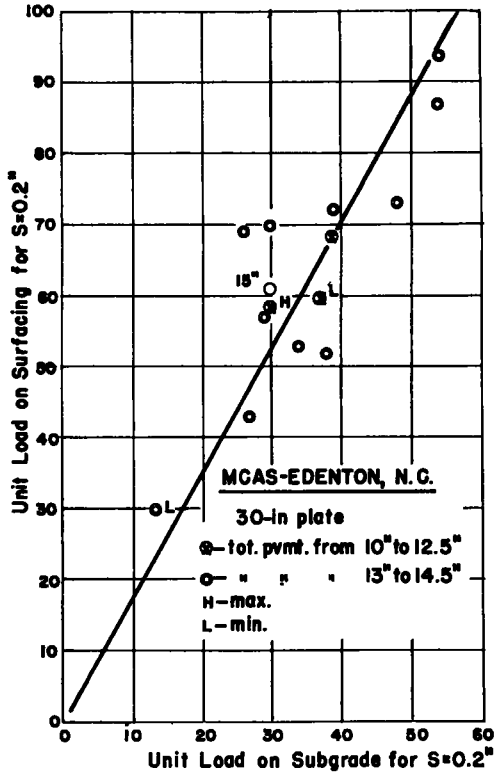


Figure 17. Relation Between Unit Leads on Subgrade and Pavement

	Avg	Max	Min
Surface			
Thickness—*in.*	4 8	6 0	4.0
Base Course			
Thickness—*in.*	8 4	10 0	6.0
Density—*lb per cu ft*	123 8	144.0	110 0
Moisture—*percent*	5 76	12 5	2 3
Subgrade			
Density—*lb per cu ft*	111 1	120 0	92 0
Moisture—*percent*	14 4	19 8	7 4

A Comparison of Theoretical and Observed Values.

In various publications the necessary pavement thickness required for a subgrade of given properties and bearing values have been computed by assuming an angle of spread of the load through the pavememt. In this

method of approach, it is assumed that the unit load on the subgrade immediately under the pavement is distributed uniformly. It is also assumed that the zone of compression is bounded by straight lines, forming the frustum of a right circular cone with the circular loaded area at the top and a larger circular area at the surface of the subgrade. According to principles of mechanics (*9*) the boundaries of the zone of compression are

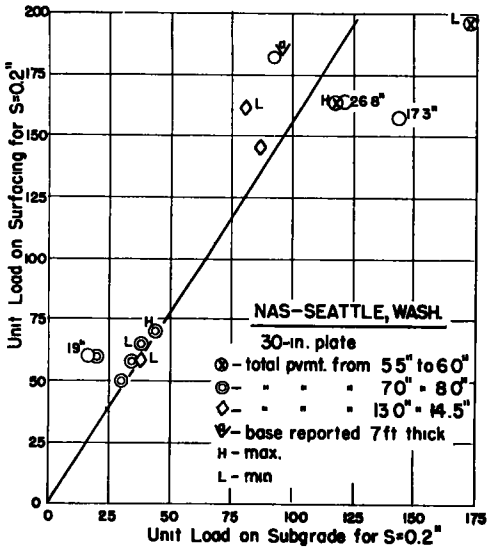


Figure 18. Relation Between Unit Loads on Subgrade and Pavement

	Avg	Max	Min
Surface			
Thickness—*in.*	4.69	7.0	3.5
Base Course			
Thickness—*in.*	6.21	23.0	0.5
Density—*lb per cu ft*	143.0	146.8	136.3
Moisture—*percent*	4.28	5.1	3.2
Subgrade			
Density—*lb per cu ft*	116.8	134.9	68.5
Moisture—*percent*	14.21	35.7	6.8

hyperbolas, not straight lines, and the unit load is distributed as a parabaloid of revolution with the highest unit load on the axis of loading at any given distance under the applied load.

The assumed angle of spread has varied from 30 to 60 degrees or more. By making the same assumption, a conical pressure distribution through the pavement and upon reaching the subgrade, there to have a uniform distribution, data typical to those plotted in Figures 19 and 20 were obtained. The angles of spread were computed in this case however since the bearing value, unit load required to deflect the pavement 0.2 in. was known. The computed angles of spread, Figures 19 and 20, are typical of numerous others not herein presented. For the Oak Grove Field, the computed angle of spread

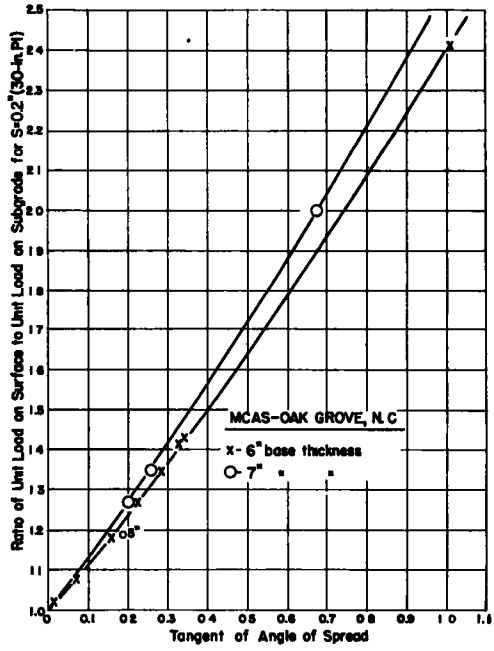


Figure 19. Spread of Load on Plate through Pavement to Subgrade

	Avg	Max	Min
Surface			
Thickness—*in.*	2.6	3.0	2.5
Base Course			
Thickness—*in.*	6.2	7.0	5.0
Density—*lb per cu ft*	124.9	142	111
Moisture—*percent*	5.0	5.9	4.0
Subgrade			
Density—*lb per cu ft*	110.9	116	103
Moisture—*percent*	6.2	11.1	2.4

varied from almost zero to an angle having a tangent of 1.0, i.e., 45 deg. For the Memphis station the tangent of the computed angle of spread varied from 0.4 to 1.07. In general, the computed tangents varied from zero to a maximum of 1.65 (58.8 deg) the bulk of the values ranging from 0.4 to 1.1. On the basis of these data assumed values greater than 45 deg would, on the average, be too

high and values as low as 20 deg may normally be expected.

If the overall modulus of compression, C_p, is obtained by loading on the pavement with one size of plate, then the unit load required to produce an equal settlement with a plate of another size may be obtained from theory (*8*).

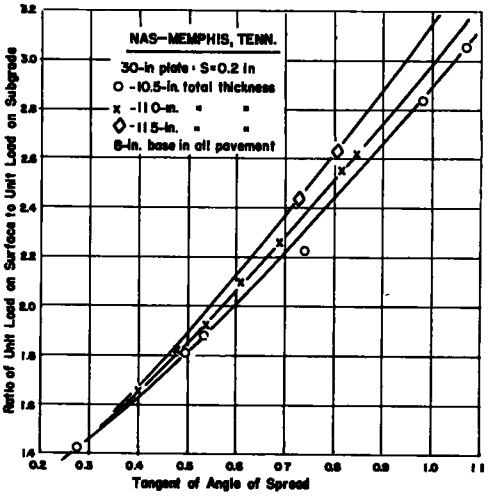


Figure 20. Spread of Load on Plate through Pavement to Subgrade

	Avg	Max	Min
Surface			
Thickness—*in.*	3.0	5.5	2 5
Base Course			
Thickness—*in.*	8 0	8 0	8 0
Density—*lb per cu ft*	119.8	134 4	110 7
Moisture—*percent*	7 2	19 0	2 4
Subgrade			
Density—*lb per cu ft*	99.8	109 7	92 0
Moisture—*percent*	19 7	23.4	11 4

The basic relations used in this theory are the following:

$$C_p = \frac{paF}{S} \quad \ldots\ldots\ldots\ldots (2)$$

$$\text{or } p = \frac{SC_p}{Fa} \quad \ldots\ldots\ldots\ldots (2a)$$

$$F = \frac{3a}{2\sqrt{a^2 + Z^2}} \quad \ldots\ldots\ldots (3)$$

F = settlement factor
Z = pavement thickness
p = unit load corresponding to any given settlement, S, and
a = radius of plate.

These expressions appear in the Manual (*3*) referred to previously. Equation (3) is valid for a Poisson ratio of 0.5 as shown previously (*8*) but for values of Z such that $\frac{Z}{a}$ exceeds 0.5 the actual value of the Poisson ratio does not affect the validity of equation (3).

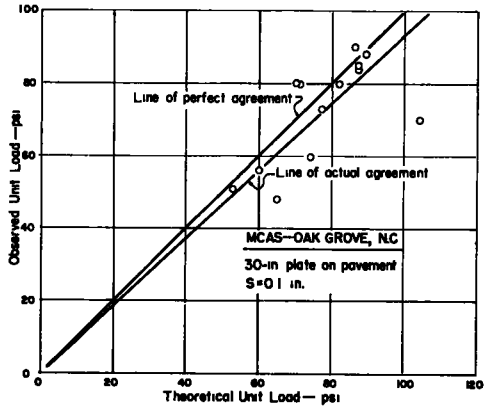


Figure 21. Agreement Between Theoretical and Observed Unit Loads Corresponding to a Given Settlement

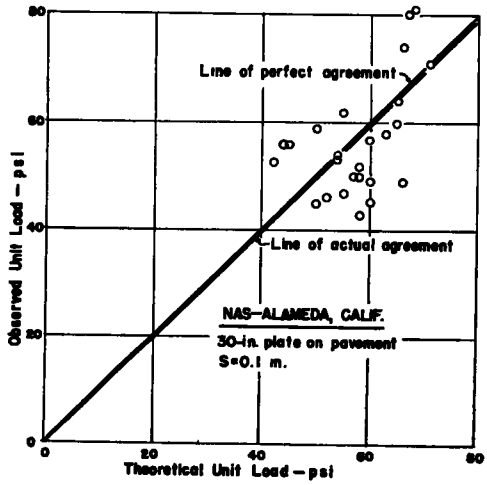


Figure 22. Agreement Between Theoretical and Observed Unit Loads Corresponding to a Given Settlement

For a given loading test on the 24-in. plate on a pavement of known thickness, Z and with F determined from equation (3), the overall modulus of compression, C_p, is computed from (2), substituting the observed unit load, C_p, of the test and the observed settlement, S, produced by that unit load.

Having computed C_p in this manner, its value is substituted in equation (2a). For the 15- and 30-in. plates the values of a in (2a) are 7.5 and 15 in. respectively. The values for F to be used in (2a) for the 15- and 30-in. plates are computed from (3), using the

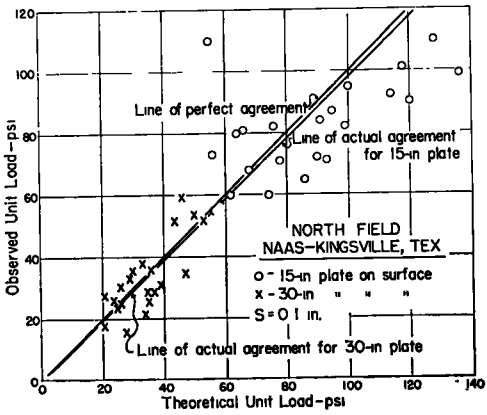


Figure 23. Agreement Between Theoretical and Observed Unit Loads Corresponding to a Given Settlement

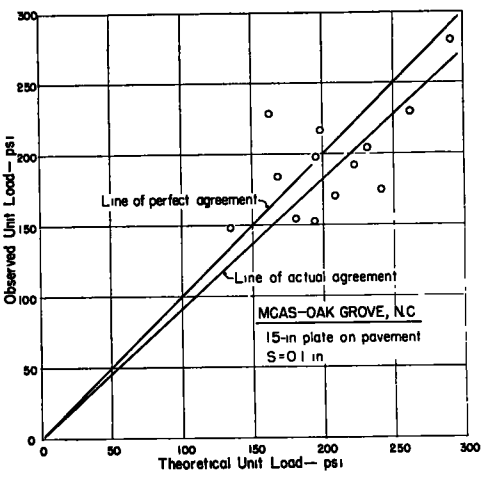


Figure 24. Agreement Between Theoretical and Observed Unit Loads Corresponding to a Given Settlement

known pavement thickness, Z, and the respective values, 7.5 and 15 in. for a. The value for S is arbitrary and fixed. For the computed values on the ordinate scale in Figures 21 to 24, inclusive, S was taken as 0.1 in. in all cases. The next step is to compute the value for p in (2a) for the 15- and 30-in. plates required to produce a settlement, S, of say, 0.1 in.

Under ideal conditions, no variations in ground or pavement within the area of the test location, the computed and observed values of p required to produce 0.1 in. settlement would be identical if the theory is correct. In this case, theoretical and computed values being the same, the plotted points would fall on a 45 deg line in Figures 21 to 24. In these figures the 45 deg line together with the line drawn through the centers of areas of the points as determined by the method of least squares, are designated as, "line of perfect agreement," and "line of actual agreement." It is observed that the divergence of the two lines is not wide but that individual points may be considerably removed from either line. In Figure 23 it is noted that the spread of individual plotted values is considerably less for the 30- than for the 15-in. plate. Still another observation is that the line of actual agreement between theoretical and observed unit loads lies consistently to the right of the 45 deg line. Thus the computed theoretical values are consistently on the safe side in these reported data.

The agreement between theoretical and measured values of unit loads producing 0.2 in. settlement, not shown here, has been found to be fully as good as the agreement shown here for a settlement of 0.1 in. This is a good indication that pavement failure is not imminent when the plate settlement is 0.2 in.

In the expressions, (2), (2a), and (3), the modulus, C_p is determined by actual loading test data and not from laboratory tests. W. H. Housel (*10*) in discussing this theoretical approach objected to the use of laboratory determined moduli in all such theoretical expressions. His objection in this connection seems to have been well substantiated by the field and laboratory data obtained during the course of this investigation.

CONCLUSIONS

On the basis of data obtained thus far in this investigation, no general or final conclusions appear to be warranted. The following conclusions are with specific reference to these data and do not extend beyond the scope of this study.

1. The subgrade moisture and density conditions found in this investigation were

considerably more favorable than the conditions that are frequently assumed in designing flexible types of pavements on the basis of California bearing ratio tests.

2. The average relative density of the top 9 in. of subgrade below the flexible pavements varied from 86 to 104 percent of the maximum density as obtained in the laboratory (the standard AASHO compaction procedure was used in the case of two of the fields reported—For all other fields the modified AASHO compaction procedure was followed). Of a total of 16 fields for which laboratory compaction tests of soils under flexible pavements were made, the relative density of the subgrade was at least 90 percent of the maximum.

3. Under the concrete pavements at six fields the average relative subgrade density varied from 79 to 98 percent of the maximum density as determined by the modified AASHO laboratory procedure.

4. For the same total load on the pavement, the settlement increased generally as the plate size decreased. For the same unit load the settlement increased as the plate size increased. These observations were in agreement with various loading test data, both published and not published, obtained by others. On this basis, the load bearing capacity of the pavement is different for different tire pressures and the higher the tire pressure, the less is the total load that the pavement can safely carry.

5. The average densities of the sand subgrades under flexible pavements at three air stations exceeded the average maximum densities obtained by the modified AASHO compaction procedure. In individual cases, the in-place density of the sand subgrade was as high as 111 percent of the maximum compacted density. It is suggested that the relatively high density of the sand subgrades can be produced by plane traffic acting through the flexible pavement to an extent that the density of the sand is increased by a combination of static load and vibration. The observed high sand subgrade densities thus tend further to indicate that under the traffic conditions and other conditions of these fields, the bearing values of the pavements were increased by the traffic.

6. The data presented showed an increase of pavement bearing value with: (1) increase of subgrade bearing value; (2) pavement thickness; (3) density of base course materials; and (4) thickness of asphalt surfacing. No effect of subgrade moisture content on the bearing value has been apparent in the analyses made thus far.

7. The computed angle of spread of load through pavement varied between zero and 58 deg with the vertical, most of the values being between 20 and 45 deg. The extent of agreement between theoretical and observed unit loads required to produce 0.1- and 0.2-in. settlement for data obtained is known to be reasonably good.

8. The data from at least 20 additional air stations are yet to be obtained. It is hoped that all analyses may be completed and that a more comprehensive report may be made at a later date.

ACKNOWLEDGEMENTS

The writer wishes to express his gratitude and appreciation for the advice and help of Captain J. C. Gebhard and Commander L. C. Coxe of the Department of Planning and Design, Bureau of Yards and Docks, in preparing this progress report. Mr. C. M. Yeomans, supervisor of the Soils Laboratory of the Bureau of Yards and Docks, rendered very capable assistance in the analyses of data.

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DISCUSSION

Mr. W. H. Campen, *Omaha Testing Laboratories:* Mr. Palmer's paper presents data relative to a number of factors involved in the designing of flexible pavements. I wish to comment on three of these factors. They are: (1) the variation of unit load bearing value with size of testing plate; (2), the strength imparting power of pavement layers; and, (3), the moisture content of subgrades.

Data pertaining to the first factor are given in Figure 11 and the relationships shown are in line with those obtained by us. However when a number of projects are evaluated in this manner it is discovered that the relationship is not constant. For instance in four of our projects the unit load bearing value obtained with an 800 sq in. plate varies from 40 to 63 percent of that obtained with a 216 sq in. plate (*1*).[1]

Data pertaining to strength imparting power of pavement layers are given in Figures 15 to 18 inclusive. In these figures the strength of the subgrade is plotted against the strength of the subgrade plus pavement and a slope line relation is established. This relationship implies that regardless of pavement thickness, the rate of strength increase per in. becomes greater as the strength of the subgrade becomes larger. By our method of evaluating the strength added to subgrades by superimposed layers this implication is not borne out. To substantiate this assertion I will use the data given in Figure 15. The example is especially appropriate because the subgrade has unit load bearing values of from 34 to 123 lb per sq in.

The data from the graph have been arranged in tabular form as shown in Table A. It will be noted from this table that each inch of pavement on the average increases the bearing value 4.75 psi when the subgrade strength varies from 34 to 37 psi. The increase is 4.0 psi when the subgrade strength varies from 61 to 98 psi, and only 3.45 psi for subgrades having strengths of 100 to 123 psi. Thus the rate of increase diminishes as the subgrade strength increases which is just the reverse of what the slope line relationship indicates.

It is not my intention to cast reflection on this type of data. On the contrary they are very much needed and when properly used can serve several purposes. For instance these four examples show that the same thickness of pavement does not have the same beneficial effect on all subgrades. For these projects, on the average the strength increase per in. of pavement thickness varies from 2.2 psi for Edenton, N. C., to 4.1 psi for Oak Grove, N. C. The information can also be used to estimate the additional pavement thickness required to accomodate larger wheel loads on a given field, or it could be used to estimate pavement thickness for new fields if the subgrade characteristics were known.

Our own experience with this type of data shows that the rate of strength increase varies from 1.2 to 4.0 psi with an 800 sq. in. plate. The high values occur on sandy subgrade and the low ones on wet plastic subgrades. Firm plastic subgrades give intermediate values. From these results it is apparent that the reaction of the subgrade is a major factor in building up pavement carrying capacity. But the subgrade reaction is not the only factor involved. The strength of the subbase, base, and wearing course is vital also. Eventually we will have plenty of data which will show that the strength imparting power of these superimposed courses depends on their

[1] Italicized figures in parentheses refer to list of references at the end of Mr. Campen's discussion.

bearing index which in turn will be a function of density and shape of particle.

Before concluding the discussion on the effect of size of testing plate on the unit load bearing value and the rate at which subgrade strengths may be increased by superimposing pavement layers, I wish to show how both of these factors may be taken into consideration on one graph. This arrangement is shown on Figure 25, p. 97, Vol. 24 *Proceedings*, Highway Research Board, which is reproduced here as Figure A. The effect of size of testing plate is shown by plotting unit load bearing value against the perimeter-area ratio of the three testing plates for the natural subgrade and superimposed layers of compacted subgrade soil, sub-base and base. To illustrate

TABLE A
STRENGTH IMPARTING POWER
Per Inch of Pavement Thickness Using 30 in Plate[a]

Load-Bearing Value		Pavement Thickness	Strength Increase		
Sub-grade	Sub-grade Pavement		Total	Per In. of Thickness	Average Values
Psi	*Psi*	*In.*	*Psi*	*Psi*	*Psi*
37	74	9.5	37	3.9	4.75
34	82	8.5	48	5.6	
69	93	9.5	24	2.5	4 0
91	128.	8 5	37	4.4	
98	140	8.5	42	4.9	
100	135	8.5	35	4.1	3.45
102	130	8.5	28	3.3	
110	140	9.75	30	3.1	
123	148	7.5	25	3.3	

[a] Data from Palmer's Report, Fig. 15.

the use of the graph I will give two examples. First let it be desired to determine the th'cknesses of superimposed layers needed to carry a 48,500-lb wheel load on a tire inflated to about 82-lb pressure. Its contact pressure would be about 90 psi and its contact area would be about 540 sq in. For a circular plate of this area $\frac{P}{A} = 0.15$. From the graph the total thickness required is 21 in. and is composed of 6-in. compacted subgrade, 12-in. compacted sub-base and 3 in. of base. For the second example let it be required to carry a 72,000-lb wheel load on a tire having a contact area of 800 sq in. The $\frac{P}{A}$ is 0.125 and from the graph it will be noted that the total thickness shown will not be sufficient. By using the rate of strength increase for the first 6 in. of base it can be calculated that 10 in. of base must be used making the total superimposed thickness 28 in.

On studying the moisture contents of the subgrades reported in this paper one is impressed with the fact that they are usually much lower than the maximums indicated by their in place voids-ratios. This means

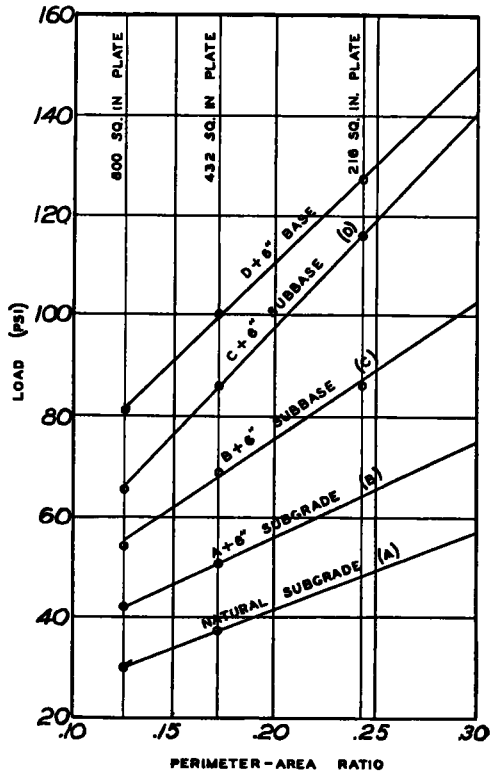

Figure A. Strength Imparting Power of Superimposed Layers, Dubuque Airport Project—Determination at 0.25-in. Total Deformation

that the soils have not taken up all the moisture they are capable of holding. In fact it appears that some of them have lost moisture judging from the low degree of saturation. If not they must have been placed in a very dry condition. In any event the data indicates quite clearly that fine grained soils of a wide variety do not become saturated by capillary moisture if the source of free water is at least about 4 ft. distant.

In connection with the subgrade moisture problem it should be pointed out that when free water is expected to come in contact with the upper portion of the subgrade special measures must be taken to prevent destructive action. There are several methods by which this can be accomplished but at this time I wish to speak briefly of our own experience with mechanically stabilized plastic soil and soil-aggregate mixtures. At the 1942 annual meeting we presented laboratory data (*2*) which showed that soils compacted to densities approximating the Proctor maximum do not take up any more moisture by capillarity than is indicated by their voids ratio. In 1943 we reported data obtained on airport runways which gave similar indications (*3*).

In finishing this discussion I wish to suggest to Mr. Palmer that he obtain as much correlation between service behavior and evaluation by the plate bearing tests as possible. If tests have been made on top of the stabilized bases as well as on top of the wearing courses this information would be welcome also. I consider the type of data assembled by Mr. Palmer to be very essential and therefore hope that he will continue the good work.

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PROFESSOR MILES S. KERSTEN, *Assistant Professor of Civil Engineering University of Minnesota*: This discussion is concerned with only one phase of Mr. Palmer's excellent paper, that of subgrade moisture conditions. Mr. Palmer has stated that the condition of the subgrade was the most important variable factor requiring study in the investigation; the writer is in complete accord with that thought. The subgrade moisture data obtained from the various Navy fields and presented in Table 1 of the paper afford a means of comparing these results with those of some previous studies, one on moisture conditions beneath highway pavements and the other those beneath airfield pavements.[1,2] In both these investigations there was a definite variation of moisture conditions with the texture of the soil, the moisture content expressed either as a percentage of the plastic limit or as a percentage of saturation being relatively low for the coarse textured soils such as sands and loamy sands, and increasingly higher for the sandy loams, clay loams, clays, and silty clays. Table 1 of Mr. Palmer's report describes the soils only by their A-group. However, the classification is sufficient for the study in mind since the general textural characteristics of the various classes are known. Most of the airports listed in Table 1 fall into one of two main groups. The first had soils described as consisting of A-1's, A-2's, and A-3's either singly or in combination; these may be considered as sandy or light textured soils; six airfields are within this group. Eight airports had soils described as A-4, A-6, and A-7 either singly or in combination; these are definitely heavier textured soils. The other eight fields apparently had a mixture of sandy and heavier textured soils since they are reported as combinations of A-2, A-3, A-4, A-6 and A-7 groups. Two fields are not included since they do not have percentage of saturation values given.

Comparing the average moisture conditions found on these three groups of fields and considering the moisture in terms of percentage of saturation the group of sandy soils had an average percentage of saturation of 35 and the highest average for any field was 44.7. On the other hand the eight fields with A-4, A-6, and A-7 soils exclusively had an over-all average percentage of saturation of 82 with a minimum average value of 63.8 and three values in the 90's. The fields with a mixture of light and heavy soils had an average of 75 with no value in the 90's. This rather general over-all view would seem to substantiate the contention that moisture values are defi-

[1] Miles S. Kersten, "Survey of Subgrade moisture Conditions", *Proceedings*, Highway Research Board, Vol. 24, (1944).

[2] Miles S. Kersten, "Subgrade Moisture Conditions Beneath Airport Pavements," *Proceedings*, Highway Research Board, Vol. 25, (1945)

nitely higher in the heavier textured soils. Although the percentage of saturation values have been used in the above comparison, a comparison on the basis of percentage of plastic limit of the soil would give, in general, similar results. The main conclusion to be drawn from this would be that if test sections are to be constructed for a new pavement design either in the field or laboratory a higher degree of saturation should be used for clay loams, clays, silty clays, and similar soils than for sands, or sandy loams.

The writer knows definitely of one design method which in effect follows this procedure, that of the Minnesota Department of Highways. In this method, the selection of a moisture content for design test is based on the plastic limit of the soil, which in their soils has been found to graduate from low values in the sandier soils to high values for the heavier classes. A higher percentage of saturation is taken for design tests for the heavier soils. This procedure is based on a extensive series of field moisture content determinations beneath pavements in service. This method of design is described in another paper presented at this meeting.[3] The data of Mr. Palmer's paper would seem to add a further substantiation to the procedure.

The data of Mr. Palmer shows that, in general, the subgrade moisture content tended to be somewhat higher under rigid pavements than under the flexible and reasons are expressed for this variation. It is agreed that it is difficult to obtain extensive data for such a comparison. It may be of interest to add therefore that in the previous study of moisture beneath airfield pavements mentioned a result similar to that of Mr. Palmer's was found. Considering only fields which had both concrete and flexible pavements, and comparing moisture contents only in similar textural classes of soils, the moisture content beneath the rigid slabs had a tendency to be greater than that beneath the flexible; on the average the difference was about 10 percentage points of saturation.

Mr. Palmer does not mention the age of the pavements on which his observations were made. If they were only a few years old it is possible that the average moisture values may increase with time.

Continual collection and analysis of data such as those presented in the paper will be a great aid in attaining a better understanding of what actually does happen under pavements and lead to more economical pavement design.

PROFESSOR D. P. KRYNINE, *Yale University:* The general idea of the important research project described in this paper is extremely simple and perfectly sound. It was decided to leave all the technical prejudices and prior opinions aside and to go to the field to investigate in place how the pavements behave under actual conditions of loading, weather, and soil surroundings. The organizations which sponsored the project, and the reporter, Mr. Palmer, are to be congratulated on the success of their enterprise.

Subgrade Moisture.—In the past few years a movement towards the study of the subgrade moisture arose and this paper represents a substantial contribution to this movement. Particularly, important field data are contained in Table 1. In the next report some additional information in this table would be desirable, however. It is stated in the paper that in no instance was the water table less than 4 ft. below the pavement. Individual data as to this depth would be welcome for the purposes of proper analysis of the table. Also a few data on the percentage of the fines in the subgrade in the column "subgrade soil type" would be of value. The lack of data referring to the time of paving is easily explained of course, by the war conditions which prevailed at that time. But even without all these data Table 1 furnishes an eloquent testimony of what may be expected under a pavement a few years after its completion. Table 4 is a logical complement to Table 1. It contains valuable information on climatic conditions; drainage conditions; and pavement conditions influencing the moisture content of the subgrade.

Degree (percentage) of saturation.—Mr. Palmer rightly states that the term saturated subgrade may mean next to nothing. In fact, a saturated soil is one in which all voids are filled with water, no matter what the value of the moisture content is. A non saturated soil may be made saturated without adding a single drop of water, just by re-

[3] J. H. Swanberg and C. C. Hanson, "Development of a Procedure for the Design of Flexible Bases," see p. 44.

ducing the voids ratio by compression. In the same way, in a general case, the degree of saturation of a soil or the percentage of the pore volume filled with water, may have no special significance. In a particular case, however, this value may be of importance. Referring to Table 1, for example, there are nine air fields of 24 in which the maximum degree of saturation is over 94 percent. Of these nine fields seven have a high average plastic limit (over 20.2), higher than in the case of any other field of Table 1; and also a high average voids ratio (over 0.630). Apparently, a high value of the degree of saturation in most cases should be explained by the presence of such soils as A-4 or A-6. Particularly, in the case of the three fields in Corpus Christi, Texas (Rodd, Cuddihy and Cabaniss) the writer shares Mr. Palmer's opinion that the subgrade in these fields became wet and soft owing to the failure of the pavement. It should be borne in mind, however, that the plastic limit of the subgrade of all these fields is high, and the permeability rather low which prevented the rain moisture from moving downward quickly.

The writer shares Mr. Palmer's opinion that the design of pavements, in a general case, should not be based on the assumption of a wet and soft subgrade. Carefulness should be recommended, however, in the case of shallow water tables and fine grained soils.

Density of Sand.—In some occassions specified in Table 3, the in-place densities of the sandy subgrades exceeded the maximum densities obtained by compaction in the laboratory. In all probability, this outstanding fact is due to the effect of plane landing and vibrations. In this connecton, Mr. Palmer's interesting discussion of the effect of vibrations on sand should be recalled (*Proceedings*, Highway Research Board, Vol. 24, (1944)).

Mr. Palmer, *Closure*. The discussions by Messrs. Campen, Krynine, and Kersten are most welcome and add materially to our fund of information in this field.

There is no constancy in the relationship of plate size and unit load bearing value of pavement as Mr. Campen points out. The relationship varies with the pavement and also with the subgrade, all other variable factors being the same. When loading directly on the subgrade, there is less variation (in unit load bearing value) with plate size than when loading on the pavement. When loading directly on very sandy subgrades, the unit bearing value changes less with respect to plate size than when loading directly on silt or clay subgrades. This writer has checked this observation repeatedly in making load tests in connection with the design of footings for structures.

It may be that too much emphasis has been given to pavement thickness per se. The fact that pavement quality and pavement construction may often be of primary importance is often overlooked. In using the CBR design curves, one finds that for a subgrade having a CBR of 3 and for a wheel load of 15,000 lb, the over-all or total presumed required pavement thickness is given as 25 in. with no account being taken of the fact that tire pressures may vary from 50 to 130 lb per sq in. However, let us ask, just what is 25 in. of pavement? It may consist of one single base course layer of 22 in. and 3 in. of asphalt surfacing. The base may have a CBR of 40 or 80. It may be densely graded crushed stone or it may be poorly graded gravel. The use of the CBR design curves, however, requires a total pavement thickness of 25 in. in either case. This seems to be a basic error in design. Certainly, 6 in. of a very superior, dense type of flexible pavement may easily have the load distributing power of 12 in. of a poorly designed pavement on the same subgrade.

On actual runways, comprising several hundred thousand sq yd in area at an air station, variations in pavement quality and subgrade bearing may easily obscure any relationship between pavement thickness and bearing value which may be observed when there are less variable conditions, that is, when the subgrade bearing and pavement quality are both constant and only the pavement thickness varies. Thus while the data of Table A shown by Mr. Campen tend to disclose an obscure relationship in the presence of several variable conditions, one questions the practical utility of the average values in the last column of his table. Probably the top 2½ in. of hot-mix asphalt surfacing accounts for a third or more of the strength increase which Mr. Campen assigns equally to each inch of the total pavement thickness. Thus a variation of ½ in. in thickness of the surfacing may be of considerably more importance than a

variation of 2 in. in the thickness of the base course and a considerable degree of variation in quality of both surfacing and base may actually outweigh all other considerations.

The approximate method of computing pavement thickness for the wheel load of 72,000 lb as illustrated by Mr. Campen presupposes a pavement quality and construction procedure that too often are not realized. Such approximations should always be checked by constructing and loading trial pavement sections. A trial pavement section, fortunately, is a check on materials and construction as well as theory. Incidentally, this writer has found that the expected linear relationship between the perimeter-area ratio and the unit load bearing value is often conspicuously missing.

From all data available it seems practically certain that the subgrades under the pavements at the fields reported were much wetter at the time of construction than at the later time when the data of Table 1 were obtained.

Professor Krynine's observations concerning degree of subgrade saturation, the effect of high water tables, the necessity for more adequate subgrade description and a study of grain size as related to moisture content are of particular value. This writer expects to follow these suggestions in completing the analysis. It should be mentioned in this connection that in the case of shallow water tables and fine grained soils, the initial subgrade conditions existing at the time of paving are most likely as unfavorable as they will be at any later time and that trial sections of pavements will yield data on loading that directly reflect the influence of the unfavorable ground water conditions. No tests in the laboratory combined with theoretical analyses can be as reliable in such cases as direct loading tests on trial sections of pavements.

Professor Kersten brings out the important point that the moisture contents are higher in the fine grained soils. In a future report it will be shown that for the moisture contents observed, the unit bearing value of the heavier textured subgrades was apparently independent of the moisture content within the limits as observed. This means, for example, that when loading on the A-4, A-6 and A-7 subgrades with the 30-in. plate, the unit bearing value was practically independent of the in-place moisture contents of these subgrades as observed. In the construction of pavement test sections, this writer's procedure has been to compact the top 6 in. or more of subgrade at a moisture content that is approximately 2 percent above the optimum as determined in the laboratory. This procedure is in agreement, apparently, with Professor Kersten's general recommendation.

Practically all of the pavements listed in Table 1 had been in service for at least 3 yr. when these observations were made. A few had been in use for well over 4 yr. It is true that the average moisture values may increase with time as Professor Kersten has considered. May it not also, however, be entirely possible that the moisture contents may remain nearly constant or decrease even further with time, especially if there is considerable traffic combined with prompt and adequate maintenance? In any case, we cannot logically assume future conditions nor can we logically decide to "beef up" the pavements on the basis of the assumption that future subgrade conditions will prove to be extremely unfavorable. Such a procedure would not be sound or scientific and it involves a factor of fear rather than any reasonable factor of safety.

TABLE B
COMPARISONS OF WHEEL LOAD RATINGS AS DETERMINED BY (A) FIELD LOADING TESTS, (FIG 11) AND (B) BY CBR DESIGN CURVES

Field	Wheel Load Rating		Maximum Wheel Load of Planes
	(a)[a]	(b)	
	lb	*lb*	*lb*
San Diego	27,500 to 46,500	20,000[b]	35,000
No. Kingsville	10,000 to 25,000	2,500	35,000
So. Kingsville	7,500 to 20,000	2,500	35,000
Chase Field	12,500 to 27,000	5,000	30,000 to 35,000
No. Whiting	20,000 to 40,000	7,500	45,000
Memphis	14,000 to 23,000	3,500	20,000
Seattle	25,500 to 50,000	7,500	45,000
Jacksonville	30,000 to 45,000	18,750[b]	60,000

[a] Wheel load rating varies with tire pressure by procedure (a).
[b] Very sandy subgrades.

It is of especial interest to compare the wheel load ratings of a few of the fields as determined by the procedure illustrated in Figure 11 with the thicknesses as computed from the CBR design curves. Accordingly, Table B is arranged. No damage by the heaviest planes at these fields has been observed.