FACTORS INFLUENCING THE LOAD-CARRYING CAPACITY OF BASE-SUBGRADE COMBINATIONS

By Jean E. Hittle and W. H. Goetz

Research Engineers, Joint Highway Research Project, Purdue University

SYNOPSIS

This paper reports an investigation of the load-carrying capacity of natural subgrade soils and base-subgrade combinations which was conducted as a war-time activity of the Joint Highway Research Project, Purdue University, under a research contract with the Technical Development Division of the Civil Aeronautics Administration.

The variables investigated included: (1) soil type, (2) type of granular-base material, (3) depth of base material, and (4) seasonal moisture. Loading tests were performed on test sections located on each of three subgrade soils (Warsaw, Crosby, and Brookston) both natural and reinforced with three depths (6-, 12-, and 18-in) of each of three base materials (pit-run gravel, clay-gravel, and crushed limestone). The influence of seasonal moisture changes was studied by loading the test sections at two seasonal moisture periods; once during the dry season and again during the wet spring season. Sixty loading tests were included in the principal part of the testing program.

One of the results of this investigation was the development of a cyclic-loading procedure with which not only the bearing values of base-subgrade combinations can be measured, but also the elastic and permanent deflection characteristics of both the base and subgrade can be determined. Although the investigation does not deal directly with pavement design, this test method has permitted an accumulation of data pertinent to an understanding of the factors controlling design.

It was found that the three subgrade soil types selected for this investigation had a wide range in load-carrying capacity which varied in accordance with their engineering soil-profile characteristics. The subgrade soil which had the least bearing-capacity also had the greatest capacity for deflection-recovery. Conversely, the subgrade soil which had the greatest bearing capacity had the least capacity for deflection-recovery. These differences in the ability of subgrade soil to recover deflection indicate the need for additional research to correlate this characteristic with design and construction procedure.

With regard to the load-carrying capacity of base-subgrade combinations, it was found that for large values of plate diameter-base depth ratio the bearing capacity depends primarily upon the characteristics of the subgrade soil; for decreasing values of plate diameter-base depth ratio the bearing capacity of the combination becomes increasingly dependent upon the stability of the base material. The results indicate that the density and the grading characteristics of the base materials are the major factors which influence base stability.

This report covers a portion of a more extensive investigation of the load-carrying capacity of base-subgrade combinations. The investigation was one of the war-time activities of the Joint Highway Research Project of Purdue University and was performed under a research contract with the Technical Development Division of the Civil Aeronautics Administration.

The paramount objective was to evaluate, by means of loading tests, some of the major factors influencing the load-carrying capacity of base-subgrade combinations. It must be emphasized that the investigation was not directed toward the development of design procedures, but rather, toward determining the relative importance of the major variables or factors influencing load-carrying capacity. It is felt that an understanding of the influence of these factors is a necessary prerequisite to the development of design procedures.

The major variables in the investigation included:

1. Subgrade Soils

2. Type of Base Material
3. Depth of Base Material
4. Seasonal Moisture.

Each of these four variables was investigated through a practical range. The scope of the study included three subgrade soil types, three types of base materials of three thicknesses each, and two seasonal moisture periods of testing.

In broad outline, the program of this investigation consisted of constructing a series of base sections, 7½ ft. square, on each of the three widely different subgrade soil types, using three types of base materials of three thicknesses each (6, 12 and 18 in.). Thus, nine base sections were installed on each of the three subgrade soil areas. Loading tests were then performed on each of these base-subgrade combinations, as well as on the three natural subgrade soils, at two seasonal moisture periods; once during the dry summer season and again during the wet spring season. Thus, with the combination of three soil types, three types of base material, three depths of base material, and two seasonal moisture periods for testing, 60 loading tests were included in the principal part of the testing program. In addition, however, a limited number of special loading tests with which base stability and subgrade deflection were investigated, were performed and are reported here.

The three subgrade soils are classified pedologically as Warsaw, Crosby, and Brookston. On the basis of general profile characteristics and extensive pavement performance surveys, these three soil types might be given a subgrade rating as follows: Warsaw—good; Crosby—fair; Brookston—poor. Representative areas of these three soil types, which were to be found in the vicinity of Purdue University, served as sites for the construction of the base sections employed in the investigation. The three soils were used in their natural state without compaction, manipulation, or treatment of any kind. For this reason, they are referred to as natural subgrade soils. The engineering soil tests on these three soils emphasize the wide textural differences in the Warsaw, Crosby, and Brookston profiles as well as the variations in texture in their individual profiles. See Table 1.

The three base materials selected were pit-run gravel, clay gravel, and crushed limestone. Since these materials are commonly employed in base-course construction, it was appropriate that they should be included in this investigation. Each of these materials was incorporated into base sections of 6-, 12-, and 18-in. thickness on the Warsaw, Crosby, and Brookston soil areas. The pit-run gravel and the clay gravel were obtained from the same local pit and were processed in a portable plant to crush the material larger than the 1-in. size. The clay content was controlled by selection from the proper horizon

TABLE 1
TEST CONSTANTS FOR SUBGRADE SOILS

Soil Type	Depth	Proctor Compaction		Liquid Limit	Plastic Limit	Plasticity Index	Finer Than No. 200 Sieve
		Max Density	Opt. M.C.				
	in.						%
Warsaw	18	116.3	13 6	35.0	18.3	16.7	57.8
Warsaw	36	117.6	12.8	0	0	0	20.1
Crosby	10	111.4	15.2	29.2	19.1	10.0	89.0
Crosby	24	114.2	14.7	32.3	16 7	15.6	48.1
Brookston	12	94.8	22.8	62.5	29.5	33.0	94.5
Brookston	20	100.2	21.2	62.5	29.8	32.7	97.0
Brookston	40	108.7	17.2	40.8	20.8	20.0	96.1

TABLE 2
SIEVE ANALYSES OF BASE MATERIALS

Base Material	Percent by Weight Passing Sieve No							
	1 in	¾ in	⅜ in.	No 4	No 10	No. 40	No. 60	No. 200
Pit-run Gravel	100	74.6	64.8	49.0	33 7	6.0	2.0	0 6
Clay Gravel	100	88.1	76.9	59 6	38.4	19.0	17.1	14.5
Crushed Limestone	100	97.5	89.2	62.4	27.1	5.7	4.1	1.7

in the pit. Other than the crushing of the over-size material, and the control of clay content by selection, no attempt was made to control the grading of these materials. The crushed limestone was a crusher-run product and likewise had a top size of 1-in. The grading analyses of these three materials are shown in Table 2.

In the construction of the base sections, it was necessary to employ hand methods for most of the program since the size of the test sections precluded the use of full-scale construction equipment. Uniform areas of the Warsaw, Crosby, and Brookston soils were first selected and the base sections, 7½ ft.

square, were "staked-out" in rows with 7½ ft. intervening between each row. The grade line for the base subgrade contact of each row of base sections was established 6 in. below the original ground line and approximated a level surface of undisturbed subgrade soil 7½ ft. wide, extending the required length of the test area. With this arrangement, the lower 6 in. of each of the base sections was "trenched" below the original ground surface.

After establishing the grade line for the base-subgrade contact, wooden forms were constructed to contain the base materials. The base materials were then placed in the forms in 2- to 3-in. lifts, each lift being compacted by hand tamping before the next lift was placed. Inasmuch as the amount of compaction that could be produced in a given base

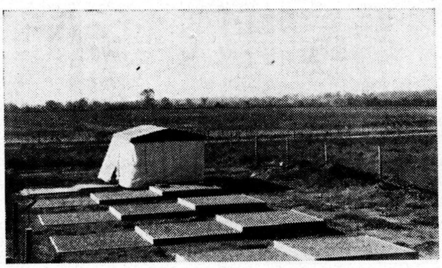

Figure 1. Layout of Base Sections in Warsaw Test Area. Testing rig with canvas canopy is shown on the far side of the test area.

material was largely dependent upon the compaction characteristics of the material itself, there was some range in the dry unit weights that were obtained for the three types of base materials.

A view of the base sections constructed on one of the soil areas is shown in Figure 1. All of the base sections were exposed to the weather without a covering or pavement surface of any kind. In this respect, as well as in area, they were dissimilar to an actual base-course installation. Because of these dissimarilities, no attempt was made to establish a relationship between material type and seasonal variations in moisture content of either the subgrade soil or the base material. It was felt that any attempt to establish such a relationship would be possible only with actual pavement installations. Moisture content determinations of both the subgrade soil and the base material were made, however, at the time of each loading test, as also was a determination of the dry unit weight of the base material. These were recorded and form a part of the test data.

METHOD OF TEST

A secondary objective of this investigation was to develop a test method which would evaluate as thoroughly as possible the variables which influence the load-carrying capacities of base-subgrade combinations. For this purpose a plate-loading technique was developed which emphasizes the compaction and rebound characteristics of the base-subgrade combination, and with which the load-deflection relationships can be obtained as well. It has several aspects which make it one of the more important developments of this investigation. From an overall perspective, however, the chief advantage of this loading technique is that it yields a maximum of fundamental test data; yet, it is practical and was applicable to the large-scale testing program of this investigation.

Specifications for the Test Procedure

The following specifications for the test procedure were adopted after a preliminary period of testing in which plate size, size of load increment, and rate of plate-deflection were all included as variables. They were designed to yield maximum information consistent with the size of the testing program. Once established, the specifications were followed in all of the routine loading tests.

In all of these routine tests a loading plate 24 in. in diameter was used. This particular size of plate was adopted since it was intermediate in size with respect to the 6-, 12-, and 18-in. bases to be tested.

Cyclic Loading: The loading technique consisted of a series of loading cycles, each of which was made up of a series of load increments and load decrements. The number of load increments and load decrements in each loading cycle was equal to the relative order of the loading cycle in the cycle series. Thus, the first cycle had one load increment and one load decrement; the second cycle had two load increments and two load decrements; etc.

Load Increments: Unit-load increments and decrements of 7.3 lb per sq in. were used in all loading tests. For the 24-in. loading plate, this unit-load increment is equivalent to an applied-load increment of 3300 lb. The choice of this particular unit-load increment was made on the basis of the time required to complete a loading test and on the number of values necessary to establish definite load-deflection relationships.

Rate of Plate Deflection: To determine the rate of movement of the loading plate, the strain dial readings were noted and recorded at 15 sec., 45 sec., 90 sec., 2 min., 3 min., 4 min., etc. after the full application of an additional load increment or decrement. In all cases the loading plate was allowed to deflect or rebound under a constant applied load until the rate of movement decreased to 0.002 in. per min or less. The next load increment or decrement was then applied and the same procedure was repeated. Again, the time required to complete a test was the basis for establishing the maximum allowable rate of plate movement. It required as much as 30 min. in some cases to reach this rate of deflection.

Cyclic-Load-Deflection Data:

The cyclic-loading curves showed that there were several significant features that the individual loading tests had in common, irrespective of the type of soil or the type or depth of base material. Figure 2, which is a loading curve typical of the data obtained in this investigation, will illustrate and supplement a discussion of these common features, and will also serve to define the terms used.

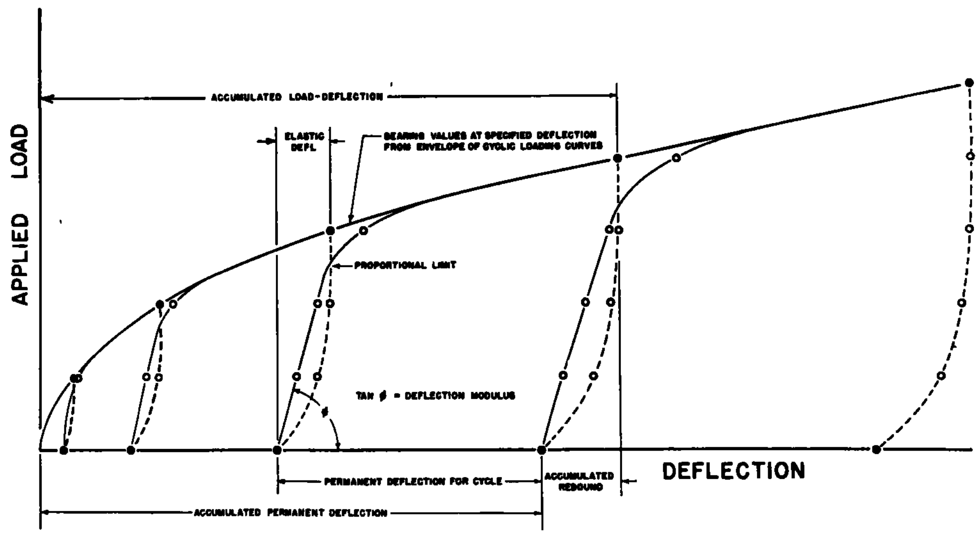

Figure 2. Typical Load-deflection Curve Obtained with Cyclic-Loading Procedure

The cyclic-loading test procedure has several desirable features. First, it establishes the load-rebound as well as the load-deflection relationships throughout the loading test. (See Fig. 2) Second, the combination of these two relationships establishes the amount of permanent deformation (Fig. 2.) that takes place as the loading test progresses. This appears to be fundamental for the analysis of load-test data. Third, the shape and slope of the loading curves provide a tangible measure of the elastic properties (Fig. 2.) o the material throughout the test. The term "elastic" is used throughout this paper in the sense that, for the loading portion of the curve, stress is proportional to strain up to a proportional limit. All of these relationships are pertinent to load-carrying capacity and their combination effects a sound and practical means of comparing load-test data. Moreover, each loading cycle of each loading test can be treated singularly or in combination with the other loading cycles of the series. This feature gives added value to the test data.

A significant feature of these and other cyclic-loading curves is the fact that, in any given loading cycle, values of deflection less than the maximum deflection of the preceding cycle are directly proportional (approx.) to the applied load, indicating that these deflections are of the elastic type. However, for values of deflection greater than the maximum deflection of the preceding cycle, the linear relationship between deflection and the applied load does not hold and the shape of the load-deflection curve indicates that these deflections are for the most part permanent. Thus, for any given loading cycle, the maximum deflection of the preceding cycle defines (approx.) the limit of proportionality between deflection and the applied load; and the proportional load-deflections define a deflection modulus. (See Fig. 2)

The counterpart to the elastic deflection in the load-deflection portion of the cycle is the rebound that ensues with the removal of load. This rebound is merely the release of elastic deflection accumulated during the load-deflection part of the cycle. The recompression of the rebound gives rise to the elastic load-deflection of the succeeding loading cycle.

Associated with these features of elastic and permanent deflections, rebound, etc., it will be noticed in Figure 2 that, as the test progresses and the relative cycle-order increases, the amount of rebound likewise increases. Also, the range of elastic load-deflection is equal to the amount of rebound of the preceding cycle. The increases in the amount of rebound therefore effect increases in the ranges of elastic load-deflection in succeeding loading cycles. Since the deflection modulus (ratio of load to deflection between no-load and the proportional limit) is essentially constant, this results in an increase in the proportional limit for successive loading cycles. These increases in rebound, elastic deflection, and proportional limit are accounted for by the additional amounts of elastic deflection gained in the interval between the proportional limit and the maximum deflection of the cycle in question.

The load-deflection portion of each loading cycle can therefore be divided into two intervals. The first interval between no-load and the proportional limit consists of elastic deflection which is the recompression of the rebound or resilient deflection of the preceding loading cycle. The second interval between the proportional limit and the maximum deflection is made up of both elastic and permanent deflections. With the removal of the applied load, the elastic deflection accumulated in both the first and second intervals becomes the total elastic rebound for the cycle. The increase in rebound over that of the preceding cycle can be attributed to the elastic deflection accumulated in the second interval, and the permanent deformation for the cycle can be attributed to the permanent deflection accumulated in the second interval of the individual load-cycle. Thus, the envelope of the individual load-cycle curves shows the accumulated load-deflections throughout the loading test and consists of both elastic and permanent deflections; the no-load deflection after rebound for each load-cycle represents the accumulated permanent deflection. Refer to Figure 2.

These features of deflection modulus, proportional limit, rebound, and permanent deformation demonstrate the fact that both natural and reinforced subgrade soils, as well as finished pavement designs, have several inherent characteristics that can be measured by means of loading tests. Actually, these features are indices of the loading-test data which can be used to evaluate the load-carrying capacities of soils and base-subgrade combinations.

BEARING VALUES

The bearing values (Tab. 3 & 4) for the dry- and wet-weather series indicate that the three subgrade soils (Warsaw, Crosby, and Brookston) have a wide range in their ability to support load. The test results also show that the load-carrying capacity of each of these three subgrade soils is in accordance with its corresponding engineering profile characteristics.

With regard to the bearing values for the base-subgrade combinations, the test results indicate that, when base materials have sufficient stability to transmit the applied loads and pressures to the subgrade soil without undergoing an appreciable amount of deformation themselves, the bearing value of the base-subgrade combination depends primarily upon the stability of the subgrade soil. In this case, increasing depths of granular base material effect corresponding increases in the bearing values of the base-subgrade combination.

In contrast to the above condition, the test results also show that under certain conditions base materials do not have sufficient stability to resist the stresses imposed by the applied loads, and therefore the bearing value of the base-subgrade combination is diminished because of the deformations occurring within the base materials themselves. Under these conditions, the bearing value of the base-subgrade combination depends on the stability characteristics of the granular-base materials, as well as the stability of the subgrade soil. In such cases, increasing the depth of the base material does not necessarily produce corresponding increases in the bearing value of the base-subgrade combination, and in several instances the test results show decreases in the bearing value for corresponding increases in the depth of granular-base materials. These relationships are representative of "base failures", occurring within the granular-base materials, which continue to be a problem for airport and highway engineers.

In view of the foregoing considerations, it is evident that a knowledge of the amount and character of deflection occurring in both the subgrade soil and the base material, under the stress of applied load, is a prerequisite for a detailed treatment of the load-carrying capacity of base-subgrade combinations. However, the test data gathered for the dry- and wet-weather series of tests recorded only the combined base and subgrade deflection, (plate deflections) caused by the loads applied to the 24-in. bearing plate, and it has not been possible in these tests to separate the deflections occurring in the subgrade soil from those occurring in the base material. However, the scope of the investigation was broad enough to show, in terms of loading-test data, the subgrade and base variables which have a dominant influence upon the load-carrying capacity of base-subgrade combinations.

TABLE 3

BEARING VALUES AT 0.5-IN. DEFLECTION AND SUPPORTING DATA FOR BASE-SUBGRADE COMBINATIONS

24-in. Diameter Plate Dry-Weather Series

Subgrade Soil	Depth	Reinforcement											
		Pit-Run Gravel				Clay-Gravel				Crushed Limestone			
		Dry Density of Base	Moisture Content of Base	Bearing Value		Dry Density of Base	Moisture Content of Base	Bearing Value		Dry Density of Base	Moisture Content of Base	Bearing Value	
		lb. per cu ft.	% dry wt.	lb.	% of subgrade	lb per cu ft.	% dry wt.	lb.	% of subgrade	lb per cu. ft.	% dry wt.	lb.	% of subgrade
	in												
Warsaw	None	105(ave) Subgrade	18.7(ave) Subgrade	14,700	100	105(ave.) Subgrade	18.7(ave.) Subgrade	14,700	100	105(ave.) Subgrade	18.7(ave.) Subgrade	14,700	100
	6	127	4.4	20,800	141	116	7.3	23,200	158	119	2.0	19,400	132
	12	121	4.9	22,600	154	113	7.0	27,000	184	118	2.0	26,500	180
	18	121	4.8	28,400	193	113	6.4	16,900*	115*	114	3.1	15,700*	107*
Crosby	None	98(ave.) Subgrade	23 1(ave.) Subgrade	14,700	100	98(ave.) Subgrade	23.1(ave) Subgrade	14,700	100	98(ave.) Subgrade	23 1(ave.) Subgrade	14,700	100
	6	136	5.6	19,200	130	106	7.5	15,800*	107*	116	3.3	14,900*	101*
	12	133	5.8	18,000*	122*	110	7 7	12,400*	84*	116	3.6	17,100*	116*
	18	128	5 9	20,200*	137*	108	8.1	9,700*	66*	114	3.8	16,400*	112*
Brookston	None	85(ave.) Subgrade	31 4(ave.) Subgrade	8,200	100	85(ave.) Subgrade	31.4(ave.) Subgrade	8,200	100	85(ave.) Subgrade	31.4(ave.) Subgrade	8,200	100
	6	132	5.4	11,200	137	123	6.4	11,800	144	123	3.8	11,300	138
	12	129	5.1	15,100	184	124	7.3	14,300	174	123	4.3	10,600*	129*
	18	130	5.1	17,500	214	122	7.8	16,000	195	118	5.8	10,200*	124*

* Bearing value indicative of base failure.

In comparing the bearing values for the base-subgrade combinations it was recognized that the following test variables had a dominant influence upon the results:

1. Subgrade soil type
2. Density of granular base material
3. Depth of granular base material
4. Type of granular base material
5. Seasonal weather variations.

Therefore, a comparison of the bearing values is made with respect to each of the above test variables under the appropriate sub-headings which follow.

In Tables 3 and 4 the bearing values at

0.5-in. deflection for the dry and wet series are shown. These values were taken directly from the envelope of the cyclic-loading curves. The value of 0.5-in. deflection is an arbitrary one; however, the same relative trends would still prevail had some other value of deflection been selected. In these two tables the bearing values for the base-subgrade combinations of the three depths (6-, 12-, and 18-in.) of each of the three granular-base materials (pit-run gravel, clay gravel, and crushed limestone) on each of the three subgrade soils (Warsaw, Crosby, and Brookston) are shown, both as a numerical value and as a percentage of that for the corresponding natural subgrade soil. In addition, the dry density and moisture content of the base material in each of the base-subgrade combinations are also shown. These two tables permit comparison of the bearing values with respect to the density, the depth, and the type of granular-base material for the respective base-subgrade combinations. Therefore, in the following sub-sections dealing with these test variables, the discussion is for the most part directed toward the data shown in Tables 3 and 4. A re-arrangement of these same data is shown in Tables 5 and 6 to facilitate the discussion in the sub-sections dealing with the effect of seasonal weather variations.

TABLE 4
BEARING VALUES AT 0.5-IN. DEFLECTION AND SUPPORTING DATA FOR BASE-SUBGRADE COMBINATIONS
24-in. Diameter Plate. Wet-Weather Series

Subgrade Soil	Depth	Reinforcement											
		Pit-Run Gravel				Clay-Gravel				Crushed Limestone			
		Dry Density of Base	Moisture Content of Base	Bearing Value		Dry Density of Base	Moisture Content of Base	Bearing Value		Dry Density of Base	Moisture Content of Base	Bearing Value	
		lb per cu. ft	% dry wt.	lb.	% of sub-grade	lb. per cu. ft	% dry wt.	lb.	% of sub-grade	lb. per cu. ft.	% dry wt.	lb.	% of sub-grade
	in.												
Warsaw	None	105(ave.) Subgrade	19 8(ave) Subgrade	14,300	100	105(ave.) Subgrade	19.8(ave) Subgrade	14,300	100	105(ave.) Subgrade	19.8(ave) Subgrade	14,300	100
	6	131	4.7	15,500	108	112	9.4	12,800*	89*	120	4.7	16,000	112
	12	127	4.6	18,400	129	108	7.8	15,600*	109*	118	2.5	19,500	136
	18	124	5.1	25,100	174	116	7.3	10,400*	73*	116	4.0	15,700*	110*
Crosby	None	98(ave) Subgrade	24 6(ave) Subgrade	10,500	100	98(ave.) Subgrade	24.6(ave) Subgrade	10,500	100	98(ave.) Subgrade	24.6(ave.) Subgrade	10,500	100
	6	134	5.6	16,100	153	110	7.6	10,100*	96*	124	4.3	13,000	124
	12	129	4 4	16,200*	154*	111	7.6	9,600*	91*	124	6.0	14,200	135
	18	126	4.5	20,700	197	110	7.5	8,500*	81*	124	3.8	21,000	200
Brookston	None	85(ave.) Subgrade	34.0(ave.) Subgrade	8,400	100	85(ave.) Subgrade	34.0(ave.) Subgrade	8,400	100	85(ave.) Subgrade	34.0(ave.) Subgrade	8,400	100
	6	126	4.1	11,600	138	119	7.8	8,900	106	123	4.9	11,100	132
	12	121	4.8	17,500	208	112	7.8	13,600	162	120	5.6	11,600*	138*
	18	120	5.2	20,800	248	113	7.9	14,400	171	116	5.1	12,200	145

* Bearing value indicative of base failure.

Subgrade Soils

It will be noted that, in the dry-weather test series, the Warsaw and the Crosby subgrade soils both had a bearing value of 14,700 lb. at 0.5-in. deflection. This value is equivalent to a unit bearing pressure of 32.5 lb. per sq. in. for the 24-in. diameter plate. The corresponding value for the Brookston subgrade was 8200 lb. or 18 lb. per sq in. A comparison of these dry-weather bearing values indicates that the bearing value of the Brookston subgrade soil is only 55 percent of that for the Crosby and Warsaw soils. This extremely low value for the Brookston soil is in accordance with its other known deficiencies as a subgrade material for airport and highway use.

In comparing the bearing values for the Crosby and Warsaw subgrade soils in the dry-weather tests (Table 3) it would seem that the Crosby soil is comparable to that of the Warsaw soil in its ability to support load, even though the Crosby soil has the less desirable profile characteristics and engineering test constants (Table 1). However, the wet-weather test series (Table 4) brings forth the deficiencies of the Crosby soil as a subgrade material and places it in its proper relative position as such. The response of the Crosby soil profile to the wet-weather conditions accounts, of course, for the reduction in the bearing value. The influence of the seasonal weather variations will be discussed presently; however, this comparison illustrates the necessity for giving the proper consideration to seasonal weather variations when making loading tests for the purpose of evaluating subgrade soils for airport and highway use.

Thus, a comparison of the bearing values in Tables 3 and 4 of the three natural subgrade soils indicates a wide range in their ability to support load, with the Warsaw soil having the highest bearing value, the Brookston soil the lowest, and the Crosby soil intermediate. Since the three subgrade soils have a wide range in bearing value, the trend in bearing values for base-subgrade combinations of a given type and depth of base material in which these soils are employed is generally the same as for the natural subgrade soils without base reinforcement. Compare Figures 3 and 6. However, because of differences in the stability of the base materials, the bearing values of the base-subgrade combinations are not always in accordance with the relative supporting power of the natural subgrade soil. Compare Figures 3 and 5.

TABLE 5
COMPARISON OF DRY-WEATHER AND WET-WEATHER BEARING VALUES FOR BASE-SUBGRADE COMBINATIONS AT 0.5-IN DEFLECTION

Subgrade Soil	Depth	Base Material								
		Pit-Run Gravel			Clay-Gravel			Crushed Limestone		
		Load								
		"Dry"[a] Bearing	"Wet"[a] Bearing	"Wet" % of "Dry" Test	"Dry"[a] Bearing	"Wet"[a] Bearing	"Wet" % of "Dry" Test	"Dry"[a] Bearing	"Wet"[a] Bearing	"Wet" % of "Dry" Test
	in.	*lb.*	*lb.*		*lb.*	*lb.*		*lb.*	*lb.*	
Warsaw	None	14,700	14,300	97	14,700	14,300	97	14,700	14,300	97
	6	20,800	15,500	75	23,200	12,800	55	19,400	16,000	83
	12	22,600	18,400	82	27,000	15,600	58	26,500	19,500	74
	18	28,400	25,100	88	16,900	10,400	62	15,700	15,700	100
Crosby	None	14,700	10,500	71	14,700	10,500	71	14,700	10,500	71
	6	19,200	16,100	84	15,800	10,100	64	14,900	13,000	87
	12	18,000	16,200	90	12,400	9,600	77	17,100	14,200	83
	18	20,200	20,700	102	9,700	8,500	88	16,400	21,000	128
Brookston	None	8,200	8,400	102	8,200	8,400	102	8,200	8,400	102
	6	11,200	11,600	104	11,800	8,900	75	11,300	11,100	98
	12	15,100	17,500	116	14,300	13,600	95	10,600	11,600	109
	18	17,500	20,800	119	16,000	14,400	90	10,200	12,200	120

[a] Values taken from Tables 3 and 4.

TABLE 6
COMPARISON OF DRY-WEATHER AND WET-WEATHER BEARING VALUES FOR BASE-SUBGRADE COMBINATIONS AT 0.5-IN. DEFLECTION

Depth	Subgrade Soil	Base Material		
		Pit-Run Gravel	Clay-Gravel	Crushed Limestone
		Load		
		"Wet"[a] % of "Dry" Test	"Wet"[a] % of "Dry" Test	"Wet"[a] % of "Dry" Test
in.				
6	Warsaw	75	55	83
	Crosby	84	64	87
	Brookston	104	75	98
12	Warsaw	82	58	74
	Crosby	90	77	83
	Brookston	116	95	109
18	Warsaw	88	62	100
	Crosby	102	88	128
	Brookston	119	90	120

[a] Values taken from Tables 3 and 4.

Density of Granular Base Material

The density of the granular-base material is highly significant in the evaluation of the influence of base stability upon the bearing values of base-subgrade combinations. This is shown by comparing the bearing values with the corresponding dry densities of the base

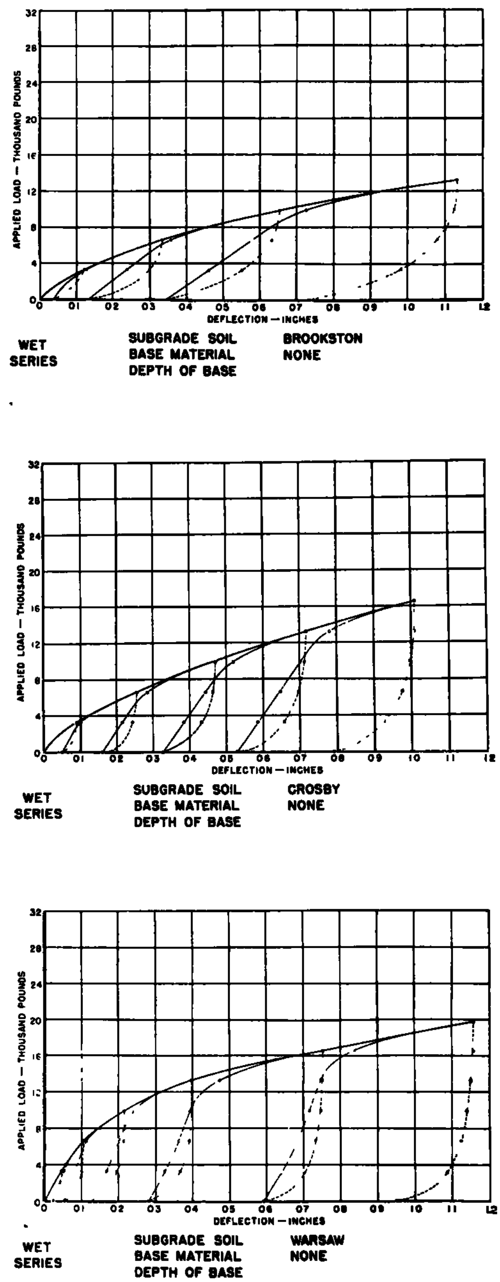

Figure 3. Loading Curves for the Three Natural Subgrade Soils Used in This Investigation. Note the wide differences in the bearing values, rebound values, and the deflection moduli for the three subgrade soils.

materials, Tables 3 and 4. In the wet and dry series of tests there are 21 base-subgrade tests out of a total of 54 in which the bearing values are indicative of excessive deformation in the base material. These bearing values have been underlined with asterisks in Tables 3 and 4. Of these 21 base-subgrade tests, it will be noted that in 15 the dry density of the base material is less than 117 lb per cu ft. Although this is an arbitrary value, it is indicated that the dry density of the base material has had a dominant influence upon stability. Having a low stability, the base materials in the above base-subgrade combinations did

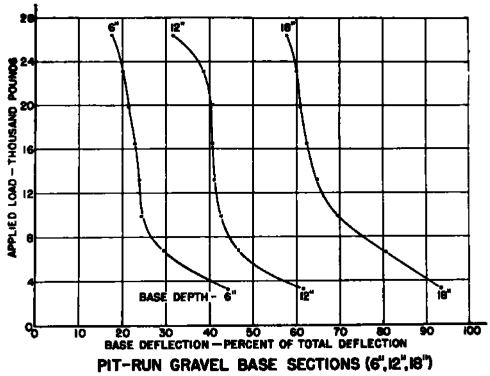

Figure 4. Base-stability Curves for Three Thicknesses of Pit-run Gravel Having a Dry Density of Approximately 130 lb per cu ft. The data for these curves were obtained with a special loading plate which facilitated an independent measurement of subgrade deflection and total plate deflection.

not offer sufficient resistance to the stresses that were imposed by the applied loads and therefore were subject to excessive deformations or base failures.

The general concept that increasing the density of base and sub-grade materials by compaction is a means of imparting additional stability or load-carrying capacity to these materials has been recognized for a number of years in the engineering literature dealing with these subjects. However, it should be emphasized that the dry density of granular-base materials is not synonymous with their stability and is therefore inadequate as a total measure of their stability. A striking example of this situation is shown by comparison of the bearing values and the corresponding dry densities of the base materials for the

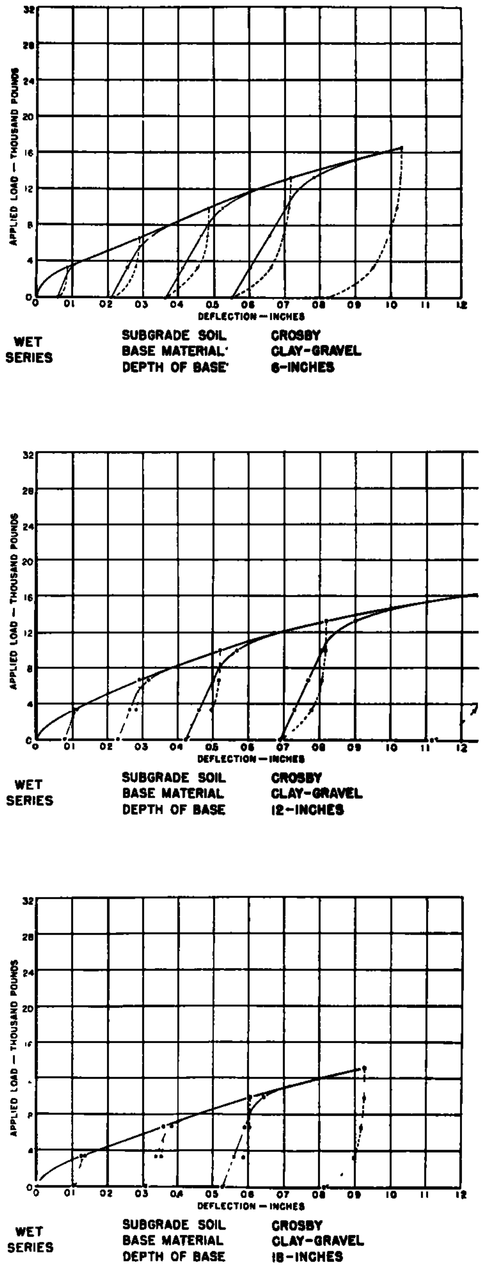

Figure 5. Loading Curves for the 6-, 12-, and 18-in Clay-gravel Base Sections on the Crosby Subgrade Soil. The decrease in bearing values for the thicker base sections indicates poor base-stability. The clay fraction in the base material, the density of the base material, and the plate diameter-base depth ratio have been contributing factors to the poor base stability.

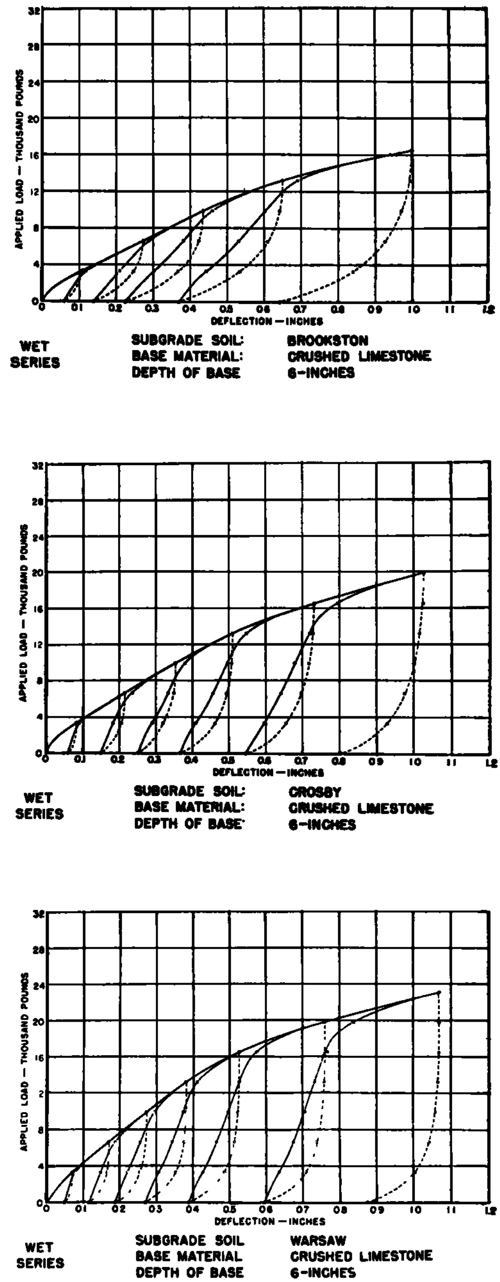

Figure 6. Loading Curves for the 6-in Crushed Limestone Base-Subgrade Combinations. A comparison of these curves with those shown in Figure 3 indicates that the rebound values and deflection moduli are primarily characteristics of the subgrade soil.

base-subgrade combinations on the Warsaw sub-grade in the dry-weather series (see Table 3). It will be noted that the dry densities of the clay-gravel bases are approximately 10 lb per cu ft less than the dry densities of the corresponding pit-run gravel bases and are slightly less than those for the crushed-limestone bases. However, the bearing values for the 6- and 12-in. clay gravel bases are notably higher than the corresponding values for bases of either pit-run gravel or crushed limestone.

Thus, the implication of these relationships would indicate that there are other factors which influence the stability of granular-base materials as much or more than the dry densities of the materials themselves. This should not detract, however, from the benefits which are realized by the proper compaction and density control of base and subgrade materials during the construction of airports and highways.

Depth of Granular Base Material

Comparison of the bearing values in Tables 3 and 4 indicates that the variable of depth of granular-base material is equally as important as the density of the base material in the evaluation of the base-stability factor. The real significance of the influence of the depth of base material upon base stability is shown by a study of the bearing values and the corresponding depths of base material for the base-subgrade combinations in which base failures occurred. Here, it will be noted that the bearing values are not only disproportionate to the depth of base material, but that there is also a tendency for the bearing values to decrease with corresponding increases in the depth of base material. The outstanding examples of this inverse relationship between the bearing value and the depth of base material are the clay gravel and crushed limestone bases on the Crosby subgrade soil in the dry series, Table 3, and the clay gravel bases on both Warsaw and Crosby subgrade soils in the wet series, Table 4.

It will be recalled that all of the loading tests in the dry and wet series were performed with a 24-in. diameter bearing plate and therefore the examples of inverse bearing-depth relationship cited above indicate that, as the ratio of the bearing-plate diameter to the depth of base material decreases, a condition exists which is conducive to a decrease in base stability. It will also be noted that the base materials associated with the inverse bearing-depth relationship have comparable densities. Therefore, it is reasonable to assume that the plate diameter-base depth ratio is an influence which has general application to the stability of base materials and which contributes to base failures as well as differences in the stability of granular-base materials that are used as a medium of reinforcement for subgrade soils.

One reasonable explanation of differences in base stability as well as base failures, which incorporates both the influence of the diameter-depth ratio and the influence of base density is that the loads applied through the bearing plate produce shearing stresses within the mass of the base material, that the base materials which had low densities likewise had a low shearing resistance, and that as the loading test progressed increasing amounts of "lateral-shoving" and compaction took place within the base material. Since all of the tests were performed with a 24-in. diameter bearing plate, the test sections having the thicker bases therefore had a greater proportion of the shearing stresses contained within the base material. Such a condition would logically be conducive to increasing amounts of base failure. The above reasoning is more or less substantiated by the test data shown in Tables 3 and 4, since it is apparent from these data that the influence of low density has been amplified by the increased depths of base materials through the influence of the plate diameter-base depth ratio.

This line of reasoning should not detract from the importance of base density as a factor influencing base stability. Actually, the influence of density of base materials and the influence of plate diameter-base depth ratio are on the same level of importance; the influence of base density involves the physical state of the base materials themselves, whereas the influence of the plate diameter-base depth ratio involves the conditions imposed by the test procedure or service use. Therefore, these two influencing factors of base stability act in combination with each other and each should be qualified by the other.

It is unfortunate that the data gathered in the dry and wet series do not permit a more comprehensive evaluation of the influence of

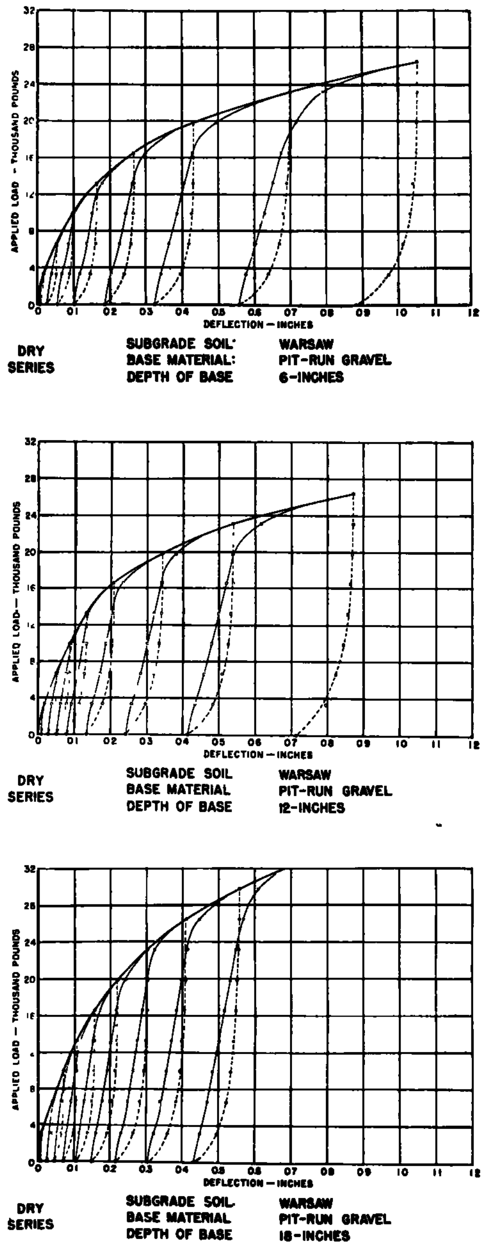

Figure 7. Loading Curves for the 6-, 12-, and 18-in Pit-run-gravel Base Sections on the Warsaw Subgrade Soil. The increases in bearing values for the thicker base sections indicate that the base materials have reasonably good stability characteristics. Note that for corresponding loading cycles the rebound values decrease and the deflection moduli increase with increases in the depth of the base material.

base depth, plate diameter, and base density in terms of base stability. In this regard, a limited number of base stability-subgrade deflection tests were performed with a specially designed 24-in. diameter plate which permitted an independent measurement of the sub-grade deflection and the total plate deflection. The purpose of these tests was to obtain data which would separate the total plate deflection into its component parts; ie, base deflection and subgrade deflection (Fig. 8).

The test procedure for these tests was the same as that for the routine tests except that deflection measurements for the subgrade were recorded concurrently with the total plate deflections. Thus, the deflection data for these tests may be divided into subgrade deflection and total plate deflection. The difference between total plate deflection and subgrade deflection is the base deflection or deformation.

A set of load-deflection curves for the 6-, 12- and 18-in. pit-run gravel base sections on the Warsaw soil is shown in Figure 4. The curves show the relation between the deflection of the base material expressed as a percentage of the total plate deflection, and the applied load. Each point on the curve represents the maximum deflection of the base material for each loading cycle. These loading curves indicate that as the loading test progresses the percentage of the total plate deflection occurring in the pit-run-gravel bases decreases. Thus as the applied loads were increased the deflections were forced into the subgrade soil.

In addition, the loading curves in Figure 4 indicate that a greater amount of deflection occurs in the thicker base sections for a given applied load and plate deflection. Therefore these special loading tests, as well as the routine loading tests of the dry and wet series, indicate that as the depth of base material increases, with respect to the diameter of the bearing plate, the base-stability factor becomes more critical, and the need for additional compaction and density control of granular-base materials in the construction of highway and airport pavements is emphasized.

Type of Granular Base Material

The influence of type of granular-base material as a test variable can also be compared on the basis of bearing values. In comparing the bearing values shown in Tables 3 and 4, with respect to the type of granular-base material, it must be recognized that the comparison is limited to one gradation of each of the three types of base material used in this investigation. Also, it must be recognized that it is difficult to interpret the bearing values with respect to the three different types of base materials, since the influence of other variables, such as base density, may obscure the effect of the type of base material. A count of the base failures in both the dry- and wet-weather series shows that there were three base failures in the pit-run gravel, eight in the crushed limestone, and ten in the clay gravel out of 18 possible in each case. The grading analysis for the crushed limestone, Table 2, shows that this material is lacking in the larger sizes of stone. This characteristic may have been a contributing influence to some of the base failures which occurred in the crushed-limestone bases. The grading analyses of the pit-run gravel and clay gravel, Table 2, show that these two materials have similar grading characteristics except for the fraction smaller than the No. 200 sieve. The clay gravel material has approximately 14 percent, by weight, finer than the No. 200 sieve, while pit-run gravel has less than one percent finer than the No. 200 sieve.

Using the test data shown in Tables 3 and 4 as an example, it will be noted that the average moisture content for the pit-run gravel is approximately 5.0 percent while the average for the clay gravel is approximately 7.5 percent. Assume that the moisture content for the fraction of the clay-gravel material larger than the No. 200 sieve is the same as that of the pit-run gravel or 5.0 percent. Under these conditions the 14 percent finer than the No. 200 sieve in the clay-gravel material contains the additional 2.5 percent of moisture, thus making the moisture content of the clay fraction approximately 23 percent.

It should be emphasized that the pit-run gravel and the clay-gravel materials were exposed to the same weather conditions and that the above example, based on the actual test data, is therefore truly representative of the influence of the clay fraction upon the moisture values for clay-gravel materials. However, the full significance of the clay fraction and its associated moisture values, lies in the fact that, at a moisture content of 23 percent, the clay fraction is in a plastic or semi-plastic state. This is highly objectionable from the standpoint of base stability, since under these conditions the plastic clay fraction tends to lubricate the whole mass of base material, thereby diminishing its shearing resistance and stability.

Although the influence of the type of base material upon base stability as reflected in

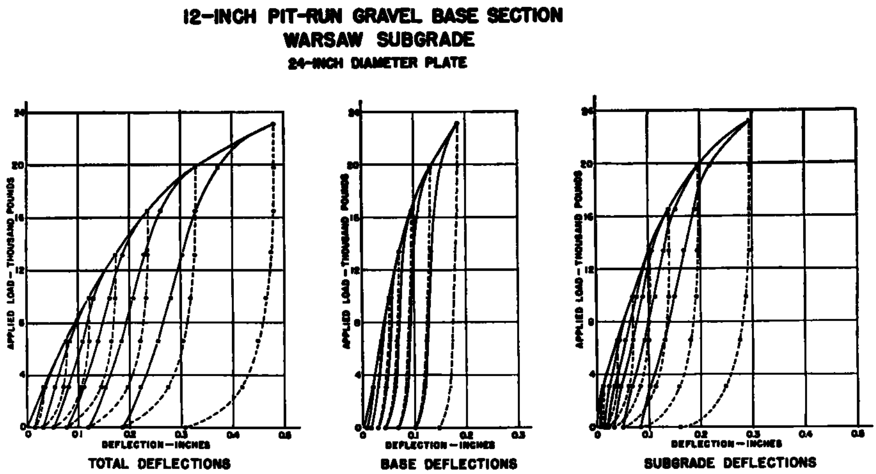

Figure 8. A set of Cyclic-loading Curves Showing the Total Plate Deflection, the Base Deflection, and the Subgrade Deflection. The data for these loading curves were obtained with the special bearing plate shown in Figure 9.

the bearing values is obscured by other influencing factors, the comparison shown in Tables 5 and 6 indicates that the pit-run gravel and the crushed limestone which were used in this investigation have comparable stability characteristics. However, comparison of the bearing values and moisture contents of the pit-run gravel and the clay gravel indicates that fine-grained (clay) materials, which have a high moisture capacity, are detrimental to the stability of granular-base materials.

Seasonal Weather Variations

The moisture or weather variable was included in the investigation by testing at two seasons of the year. It will be recalled that the test data gathered in connection with the wet-weather test series represent a retesting of the same base-subgrade combinations which had been previously tested in connection with the dry-weather test series. Therefore, in making a comparison of the bearing values in the dry series with the corresponding bearing values of the wet series, it must be recognized that in addition to the influence of the seasonal weather variations, the bearing values of the wet-weather series may also reflect the influence of any "residual-effect" remaining from the previous loading tests of the dry-weather series. It should also be noted that the tests on the subgrade soils without reinforcement were made on new locations for both series, and, therefore, these tests do not include the effect of retesting.

Comparison of the wet- and dry-weather bearing values for each of the three subgrade soils alone (Table 5) shows the influence that seasonal weather variations have upon the bearing values of the respective subgrade soils. In the cases of the Warsaw and the Brookston subgrade soils, the seasonal weather variations had little if any effect on the bearing values, which may be attributed to their respective horizon developments and profile characteristics. Both the Warsaw and the Brookston soils have profiles in which the arrangement of the horizon developments are such that there is a gradual transition in the textural and physical properties from the surface horizon to the unweathered parent materials. In addition, the Brookston soil profile is poorly drained, and by virtue of its low natural dry-density value, has a high moisture capacity even during the dry-weather season, while the Warsaw soil profile has good drainage, a higher natural dry-density value, and therefore a low moisture capacity at all times regardless of weather conditions. The arrangement of horizon developments and the profile characteristics of the Warsaw and the Brookston soils accounts for, or at least seeks to explain, why the seasonal weather variations had little influence on the bearing values for these two subgrade soils.

In contrast to the Warsaw and the Brookston soils, the Crosby soil has a bearing value for the wet-weather test which is considerably less than that for the dry-weather test. In a like manner, this reduction in bearing value can be explained by the arrangement of horizon developments and profile characteristics of the Crosby soil. In this soil profile the arrangement of the horizon developments is such that there are extreme contrasts in the textural and physical properties throughout the soil profile. Texturally, the "weathered" horizon of the Crosby profile is a silty-clay with the silt predominating. Between the "weathered" horizon and the parent material (glacial till), however, is a "clay-pan" development in which a predominately clay texture prevails. These textural contrasts in the horizon developments are conducive to a "water-logged" profile during wet-weather seasons, which in combination with the textural contrasts themselves explain, partially at least, the reduction in the wet-weather bearing value for the Crosby subgrade soil. This result is in agreement with the experience that highway engineers have had with the performance, during the spring season, of secondary highways located on Crosby or Crosby-like soils.

The seasonal weather variations also had a pronounced influence on the bearing values of the base-subgrade combinations. This is particularly true of the clay-gravel bases since the comparison shown in Table 5 indicates that without exception the clay-gravel bases underwent the greatest reduction in bearing value with seasonal moisture changes.

A study was made of the moisture contents for corresponding pit-run gravel and clay-gravel base sections in the dry- and wet-weather series. (See Tables 3 and 4.) It was found that for the dry-weather series the average difference between the moisture contents of corresponding pit-run gravel and clay-gravel base sections was 2 percent; for the "wet-weather" series the average difference was 3 percent. With a clay fraction of 14 percent in the clay-gravel material, these data indicate that the respective average moisture contents of the clay fraction would be 20 percent and 27 percent. This increase in moisture in the clay fraction would cause a corresponding increase in the plasticity of the clay fraction, which in turn would decrease the stability of the clay-gravel base sections as shown by the reduction in their bearing values.

With regard to the reduction in bearing value of the pit-run gravel and the crushed-limestone base sections in the wet-weather tests as compared to the dry-weather tests, the amount and number of reductions in either case is approximately the same. This would indicate these two types of base materials respond to the influence of weather variations in about the same degree.

In Table 5 it will be noted that in several instances the wet-weather bearing values were higher than the corresponding dry-weather bearing values. Although it has not been possible to completely explain the increases for the wet-weather bearing values, it is logical to assume that the "residual-effect" of the previous testing may have been a contributing factor.

In addition to the influence of seasonal weather variations, the stabilities of the base materials and the relative decreases for the "wet" bearing values have also been influenced by the relative supporting power of the three subgrade soils. The arrangement with respect to soil type of the bearing values for the dry and wet series, Table 6, shows a rather significant trend in the ratios of "wet" to "dry" bearing values. It will be noted that, with but one exception, the ratios of "wet" to "dry" bearing values for each depth-type combination are the least for the Warsaw soil, the largest for the Brookston soil, and intermediate for the Crosby soil.

From the standpoint of base stability, the trend of the ratios of the "wet" to "dry" bearing values shown in Table 6 would seem to indicate that base materials have the least stability on the Warsaw subgrade soil and have the greatest stability on the Brookston subgrade soil. While this is a true statement under the conditions of the comparison, it must be recognized that the comparison is based on a ratio of bearing values and therefore does not include the actual bearing value. The true significance of the comparison shown in Table 6 lies in the fact that the bearing values of the Warsaw base-subgrade combinations are higher than those on the other two subgrade soils, and therefore the base materials placed on this soil are more vulnerable to the forces producing instability. In contrast to the above conditions, the Brookston base-subgrade combinations have the lowest bearing values and the base materials placed on this soil are least vulnerable to the conditions of instability. Therefore, from the standpoint of bearing value, the base-stability factor is the most critical for the base materials on the Warsaw subgrade and the least critical for base materials on the Brookston subgrade; conversely, the subgrade factor is the most critical for the Brookston subgrade and least critical for the Warsaw subgrade.

DEFLECTION CHARACTERISTICS

In addition to the wide range in the bearing values of the Warsaw, Crosby, and Brookston soils, the cyclic-loading curves for the three subgrade soils demonstrated that there is also a wide range in their deflection moduli and rebound values. Both of these deflection characteristics may be significant factors in the evaluation of base-subgrade combinations Also, the rebound of a subgrade soil is pertinent to the construction of base courses since experience has shown that the compactibility of the base material is influenced by the rebound characteristics of the subgrade soil on which it is compacted.

The straight-line portion of the cyclic-loading curves also has significance to the evaluation of load-carrying capacity, since it indicates that the load-deflections of base-subgrade combinations can be limited to the elastic type if the previous maximum deflection is not exceeded. Also, since elastic deflections which cause little or no permanent deflection upon the removal of load, are the ultimate goal in design procedures, the straight-line portion of the cyclic-loading curves suggests the need for additional research to correlate, if possible, this feature of the cyclic-loading test with construction procedures used in the handling of subgrade and base-course materials.

Rebound Values

The loading curves in Figure 3 show that for a given loading cycle the Brookston soil has the greatest amount of rebound; the Warsaw soil, the least; and the Crosby soil an intermediate rebound value. Further comparison of the loading curves for the natural subgrade soils in Figure 3 and the loading curves for the crushed-limestone base sections in Figure 6 shows that the rebound characteristics of the base-subgrade combinations depend primarily upon the rebound characteristics of the subgrade soil. Thus, the relative amount of rebound for comparable base sections on each of the three subgrade soils is the same as for the natural subgrade soils without reinforcement.

TABLE 7
SUMMARY OF REBOUND VALUES FOR THIRD LOADING CYCLE
Values Expressed in Inches

Subgrade Soil	Reinforcement									
	None	Pit-Run Gravel			Clay Gravel			Crushed Limestone		
		Depth (in)								
		6	12	18	6	12	18	6	12	18
Dry Series										
Warsaw	.063	.043	.036	025	.046	.034	.031	053	.047	.044
Crosby	.091	.073	051	038	081	.070	062	.082	.057	.050
Brookston	.311	198	.091	062	182	123	0.54	178	.152	.103
Wet Series										
Warsaw	085	.076	.064	.043	.080	058	.060	.085	.071	057
Crosby	.143	089	.075	.050	.122	.097	078	.102	079	056
Brookston	310	.193	119	071	240	.151	.101	.211	160	.109

The amount of rebound for the base-subgrade combinations is also influenced by the depth of the base material, since the subgrade loadings are distributed over a greater area for the thicker base sections. Reference to the loading curves for 6-, 12-, and 18-in. pit-run-gravel base sections in Figure 7 shows that increases in the depth of base are accompanied by corresponding decreases in the amount of rebound for a given loading cycle.

The rebound values for the third loading cycle of each of the loading tests of the dry and wet series are shown in Table 7. Comparison of these values also shows the influence of depth and soil type upon the amount of rebound. It is significant to note that, in general, the rebound values for the wet series are slightly higher than for the dry series. These higher rebound values for the wet series can logically be attributed to the change in moisture conditions; likewise, a decrease in the spreading of the load by the base materials (i.e., a decrease in the loaded area of the subgrade) also may have been a factor.

The data obtained in a limited number of exploratory tests performed with a specially designed bearing plate (see Fig. 9), which permitted an independent measurement of subgrade deflection and total plate deflection, indicate that a small portion of the total rebound for a given loading cycle must be attributed to the rebound of the base materials themselves. A set of loading curves obtained with the special bearing plate is shown in Figure 8. Although this example is for a specific case, the relative shape of the loading curves is typical for these special tests. Comparison of the subgrade deflection curves and the total plate deflection curves obtained in these tests again emphasizes that the rebound of the base-subgrade combination is primarily a characteristic of the subgrade soil. However, it will be noted that there is some rebound in the base deflection curves, which for the most part is confined to the last load decrement of the loading cycle.

The few tests that were performed with the special bearing plate also indicate that the amount of rebound occurring in the base material is independent of the rebound characteristics of the underlying subgrade soil; also, that the amount of base rebound increased with corresponding increases in the applied load and the depth of base.

From one viewpoint, the rebound values of the base-subgrade combinations more or less complement the corresponding bearing values since in many instances the ability of a base course to recover from load-deflection may be a significant factor in its performance. Likewise the rebound characteristics of a subgrade soil are a pertinent factor in the construction of base courses. The significance of rebound in this instance lies in the relationship of the compactibility of the base material and the rebound characteristics of subgrade soil on which it is compacted.

Those familiar with the construction of base courses will fully appreciate the difficulties encountered in the compaction of granular-base materials. With present equipment and design procedures and under the most desirable field conditions, it is difficult to obtain the proper amount of compaction in granular-base materials. The most frequent difficulty encountered is the "wave-action" which is produced in the base material itself in advance of the rolling or compaction equipment as it moves forward. Naturally, when subgrade soils are encountered in the field which have low bearing values this "wave-action" in the base material is accentuated. Add to this condition the high rebound values of a subgrade soil similar to the Brookston which will recover approximately 50 percent of the load-deflection, (see Fig. 3) and the net effect is that a limiting density of the base material may be reached beyond which additional compactive effort is not effective in producing increases in density. Although the study of the compactibility of base materials by field-construction methods was not within the scope of this investigation, the relative rebound values for the Warsaw, Crosby and Brookston soils are in general agreement with

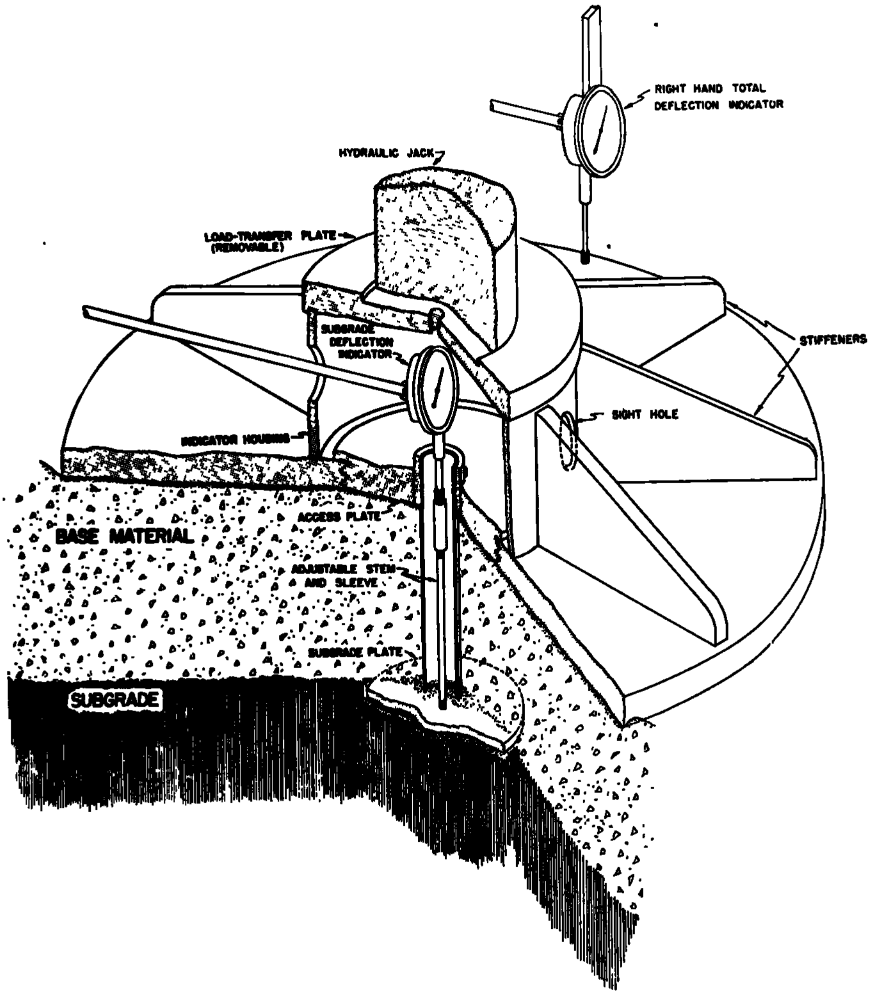

Figure 9. Schematic Diagram of Special 24-in Bearing Plate for Measuring Subgrade Deflection and Base-stability

the degree of compaction which highway engineers have usually found obtainable in granular base courses when these or similar soils have been traversed by highway locations.

Deflection Moduli

The relation of the deflection modulus to the cyclic loading curves has been discussed previously under "Cyclic Load-Deflection Data" in the section on method of test; briefly, however, the deflection modulus is the ratio of the applied load to deflection (i.e., slope of loading curve, see Fig. 2) in that range of deflection between "no-load" and the proportional limit for the cycle. The maximum deflection for the preceding cycle approximately defines the proportional limit for the proportional load-deflections; likewise the amount of rebound approximately defines the range of proportional load-deflections. The deflections which define the deflection modulus are referred to as elastic deflections since they are proportional to applied load and cause little or no permanent deflection with the removal of applied load.

The loading curves for the natural subgrade soils shown in Figure 3 indicate that the Brookston soil has the lowest deflection modulus, the Warsaw soil the highest, and the Crosby soil an intermediate deflection modulus. It is significant to note that the relative deflection moduli for the Warsaw, Crosby, and Brookston subgrade soils are in accordance with their respective engineering characteristics (see Table 1).

A further comparison of the loading curves for the natural subgrade soils (Fig. 3) and the loading curves for the crushed limestone base sections (Fig. 6) shows that the deflection moduli as well as the rebound values are primarily characteristic of the subgrade soil. It is significant to note that the subgrade soils and base-subgrade combinations which have high rebound values likewise have low deflection moduli; conversely, low rebound values are accompanied by high deflection moduli. It is therefore more or less a general proposition that the deflection moduli obtained by the cyclic-loading procedure are inversely proportional to the amount of rebound.

The relation between the deflection moduli for the base material and for the subgrade soil is shown by the loading curves obtained with the special bearing plate (Fig, 9). In contrast to the subgrade-deflection curves which have "looping" characteristics, it will be noted that the base-deflection curves are quite similar to the rebound curves of the preceding loading cycle. Likewise, it will be noted that the deflection moduli for the base materials are high in comparison with the deflection moduli for the subgrade soil.

The deflection moduli for the third loading cycle of each of the loading tests in the dry and wet series are shown in Table 8. A comparison of these values shows that the depth of base material, as well as the subgrade soil

TABLE 8
SUMMARY OF DEFLECTION MODULI FOR THIRD LOADING CYCLE
Values Expressed in 1000 lb. per in.

Subgrade Soil	None	Reinforcement								
		Pit- Run Gravel			Clay Gravel			Crushed Limestone		
		Depth (in.)								
		6	12	18	6	12	18	6	12	18
Dry Series										
Warsaw	155	220	275	400	230	300	300	180	200	250
Crosby	100	135	210	390	130	200	180	115	200	215
Brookston	35	60	120	200	60	95	210	60	65	110
Wet Series										
Warsaw	115	115	140	240	100	165	160	105	150	165
Crosby	60	115	140	190	70	95	105	90	115	170
Brookston	30	55	90	180	40	70	115	45	65	95

type, has a controlling influence on the deflection moduli for the base-subgrade combinations. The increase in the deflection moduli for the greater thicknesses of base material is, of course, attributed to the corresponding increases in the loaded area for the subgrade soil. Accordingly, variations in the deflection moduli for comparable depths of base material are attributed to differences in the ability of the base materials to distribute the load to the subgrade (i.e., differences in the loaded area of the subgrade). The seasonal weather variations also have had an influence on the deflection moduli since it will be noted that the values for the wet series are lower than the values for the dry series.

The loading curves in Figures 2 and 3 indicate that the proportional limit for a given loading cycle occurs at the maximum deflection of the preceding loading cycle. The proportional limit of a given loading cycle is also defined (approximately) by the deflection modulus for the cycle and the rebound value for the preceding loading cycle. Thus the proportional limit for a loading cycle can be defined in terms of either applied load or deflection. In terms of applied load, the proportional limit for a loading cycle is usually between 80 and 95 percent of the maximum load of the preceding loading cycle.

These relationships are significant since they establish, in terms of the cyclic-loading test at least, the amount of applied load necessary to produce a given amount of elastic deflection for a particular base-subgrade combination. Since it is desirable to limit the load-deflections of base-subgrade combinations to the elastic type which cause little or no permanent deflection upon the removal of load, the relationship of deflection modulus, proportional limit, and rebound suggest a possible application in this direction.

Assuming that these relationships between the deflection moduli, the proportional limits, and the applied loads are directly applicable to field and service conditions, one practical application that could be made of them would be in the construction phase of either airports or highways. Consider a hypothetical example of an anticipated service load of 10,000 lb, a deflection modulus of 100,000 lb per in., and a proportional limit of 85 percent. The 85 percent proportional limit indicates that an applied load of 11,800 lb $\left(\frac{10,000}{0.85}\right)$ is required to insure that the service load would cause only elastic deflections. With the deflection modulus of 100,000 lb per in., the service load of 10,000 lb will therefore cause a deflection of 0.10 in. The implication made here is that the construction equipment used for compacting the subgrade should have sufficient weight, size, etc., to produce a condition in the subgrade soil equivalent to that produced by the 11,800-lb applied load. Admittedly, the above is a hypothetical example, since the correlation between load-deflection of loading tests and that of traffic loading is not accurately known. However, the established relationships of deflection moduli, proportional limits, and applied loads as they occur in the cyclic-loading tests clearly show the need for additional research so that these relationships may be correlated with construction procedures used in the handling of subgrade soil and granular base materials.

SUMMARY OF RESULTS

It is hoped that the results of this investigation will prove fundamental and useful to those responsible for design of highway pavements and airport runways. Although the development of design procedures was not an objective of this investigation, it is felt that the method of test which was developed and adopted has permitted an accumulation of data pertinent to an understanding of the factors controlling design.

From the loading tests conducted on the simulated base-course installations with rigid bearing plates, the following important results were indicated:

1. The load-carrying capacity of base-subgrade combinations depends on (a) the subgrade-soil characteristics, (b) the base-stability characteristics, and (c) the depth of base material. For base-subgrade combinations having good base stability, the load-carrying capacity depends primarily upon the subgrade soil. In this case, increasing depths of base material give corresponding increases in the load-carrying capacity for the base-subgrade combination. In contrast to the above, poor base stability will diminish the load-carrying capacity of the base-subgrade combination because of deformations occurring within the base materials themselves. In such cases, increases in the depth of base material do not necessarily give increases in the load-carrying capacity for the base-subgrade combination.

2. The base-stability characteristics (or the ability of the base materials to transmit the applied loads to the subgrade without undergoing deformation) are influenced by (a) the density of the base material, (b) the type of base material, and (c) the plate diameter-base depth ratio.

Proper density of the base material is necessary for good base stability; otherwise, the applied loads will produce deformations in the base. The results of tests on the clay-gravel bases indicate that fine-grained material with a high-moisture capacity is a detriment to good base stability. For increasing values of the plate diameter-base depth ratio, a greater proportion of the total stress is contained by the base materials themselves. For this reason, the greater the depth of base in relation to the size of the loaded area, the more critical becomes the factor of base stability.

3. Seasonal weather variations must be considered in the evaluation of the load-carrying capacity of base-subgrade combinations. Although the difference between the moisture values during the dry-weather season and during the wet-weather season was not great, there was an overall downward trend in the bearing values for the wet-weather test series. The seasonal weather variations influence the stability of the granular base materials as well as the stability of the subgrade soil. The amount of the seasonal-weather influence upon load-carrying capacity of the subgrade soil depends upon the textural characteristics of the soil profile.

4. The cyclic-loading technique which was developed in connection with the method of test emphasizes the load-rebound as well as the load-deflection characteristics of the materials being tested. The rebound values as well as the deflection moduli values of the loading curves for base-subgrade combinations are primarily subgrade soil characteristics. The subgrade soils and base-subgrade combinations which have low deflection moduli likewise have high rebound values; conversely, high deflection moduli are accompanied by low rebound values. The base-subgrade combinations have higher deflection moduli than the natural subgrade soil since the base materials distribute the loads over a greater area of the subgrade. Likewise, increases in the depth of base produce greater deflection moduli through increases in the loaded area of the subgrade soil.

5. The relationships established by the cyclic load-deflection curves indicate that the load-deflections of base-subgrade combinations can be limited to the elastic type if the previous maximum deflection is not exceeded. Since the range of elastic deflection varies with the subgrade-soil characteristics, the depth of base material, and the applied load, the load-deflection relationships of the cyclic-loading test suggest the need for additional research so that these relationships may be correlated with design and construction procedures used in the handling of subgrade soil and granular-base materials.

APPENDIX

TESTING EQUIPMENT

One of the preliminary, but important, phases of this investigation was the development of testing equipment with which to perform the loading tests. This included the design and construction of a mobile anchorage rig and a series of steel bearing plates; also, the procurement of hydraulic jacks and strain dials was necessary. In developing the testing equipment, considerable emphasis was placed on mobility and flexibility so that the equipment could be easily adapted to a variety of field conditions and test procedures.

Mobile Anchorage Rig

The design and construction of an anchorage rig to supply the load reaction for the loading tests was one of the major equipment developments of this investigation. The anchorage rig which was developed has the desirable feature of being mounted on wheels, thus making this relatively heavy piece of equipment mobile. The hitch and steering apparatus make it possible to tow the anchorage rig behind a truck as a trailer is towed. This feature of mobility proved to be quite expedient in the testing operations.

To provide anchorage for the anchorage rig, deadmen anchors were adopted. These anchors were made by attaching a loop of ⅝-in. wire cable to the center of a railroad tie 7½-ft long. The deadmen were installed approximately 5 ft below the ground surface with the cable loop extending a few inches above the ground surface. Under test conditions a deadman anchor was attached to each corner of the anchorage rig.

Bearing Plates

The circular bearing plates used in this investigation are made of hot-rolled steel and are machined on all surfaces to a minimum thickness of 1 in. The diameters of the five bearing plates which make up the series vary from 12 in. to 30 in. by 6-in. intervals. The top surface of each plate is recessed so as to receive the next smaller plate. Thus, in testing with the 24-in. diameter plate, for instance, the 18- and 12-in. diameter plates are also used, each plate being placed concentric with and on top of the next larger plate. This arrangement serves the purpose of stiffening the testing plate and thereby alleviates the bending stresses in the testing plate when high bearing pressures are encountered.

Special Bearing Plate

A special 24-in. diameter bearing plate was developed for studying subgrade deflection and base-stability. This plate measures subgrade deflections independently of the total plate deflections. A schematic diagram of this special bearing plate is shown in Figure 9. The plate proper consists of a 24-in. diameter steel plate, 1 in. thick, with a removable access-plate $5\frac{1}{4}$ in. in diameter, concentric with the center of the plate. The dial or indicator housing for the subgrade deflection indicator consists of an 8-in. steel pipe section, $\frac{3}{8}$-in. wall thickness, 6 in. in length, welded to the top face of the plate proper concentric with the center of the plate. Two holes, 2 in. in diameter, in the side of the indicator housing and diametrically opposite facilitate the viewing and the support of the subgrade deflection indicator. To give added stiffness and rigidity to the assembly, eight radial stiffeners of $\frac{1}{2}$-in. steel are welded to the plate and the pipe section at an equal spacing around the periphery of the plate. A steel plate with a minimum thickness of 1 in., recessed on one face to receive the indicator housing and recessed on the opposite face to receive a 50-ton hydraulic jack, provides the load-transfer between the hydraulic jack and the bearing plate. Other pertinent features of this special bearing plate are the 5-in. diameter subgrade plate and a series of adjustable stems and sleeves which facilitate the connection between the subgrade plate and the subgrade deflection indicator.

Hydraulic Jack

A hydraulic jack with a 50-ton load capacity was used to apply the loads to the bearing plates. The jack consists of a ram and hand pump connected by a flexible hose. Oil pressure is supplied to the ram through the flexible hose by means of the hand pump. A pressure gauge included in the system measures the applied load. This loading arrangement proved to be both practical and convenient.

Equipment Set-Up

In making the equipment setup, the anchorage rig was first moved into place over the base section to be tested and connected to the anchor cables by means of adjustable clevis connections. The anchor cables were then stressed by means of hydraulic jacks placed at either end of the anchorage rig and the rig brought into a level position by adjusting the length of the clevis connections. The next step in setting up the equipment was to seat and level the bearing plate on a thin layer of sand covering the surface of the base section. With the smaller bearing plates of the series in place, a hydraulic jack equipped with a spherical bearing head was inserted between the anchorage rig and the bearing plates. A seating load of approximately 1 lb per sq in. was applied to the bearing plates for a short time and then released.

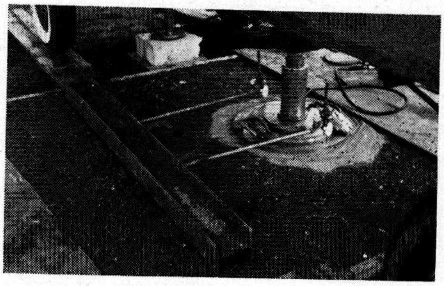

Fig. 10. Equipment Set-up Showing Arrangement of Bearing Plates and Deflection Dials

The last step of the equipment setup was to install strain dials. In this operation a structural steel H-section, 6 ft long, was seated with flanges vertical in the surface of the soil or granular material approximately 30 in. from the center of the bearing plate. The dial support was then clamped to the H-section, the dials mounted at either side of the bearing plates, and the proper adjustments made. See Figure 10.

The procedure for setting up the equipment for the tests in which the special 24-in. bearing plate was used was the same as above except for the installation of the subgrade plate. Installation of the subgrade plate was made after the plate assembly had been brought into a level position and a seating load applied. In this operation the load-transfer plate and the access plate were removed from the assembly (see Fig. 9) and the volume of base material beneath the area of the access plate was removed. The exposed subgrade soil was then covered with a thin layer of fine-graded sand to insure a uniform bearing of the subgrade plate. The subgrade plate with an adjustable stem of the appropriate length was then placed on the sand cushion and brought into alignment with the rest of the bearing plate assembly by manipulation. The subgrade bearing plate, as well as the remainder of the area of the hole through the base material, was then covered with a layer of fine sand to a depth of approximately ¼-in. This layer of sand was a precautionary measure to insure uniform bearing of the base material upon the top of the subgrade plate. A sleeve of steel tubing of the appropriate length and size to make a connection with the access-plate was then placed on top of the subgrade plate concentric with the stem fixture. The base material which had been previously removed was then carefully recompacted in the hole. Special care was necessary in this operation to prevent disturbing the alignment of the subgrade plate and the concentric position of the sleeve and to replace the base material to its original density. The access plate was then screwed into position (See Fig. 9) and the lower 2-in of the adjustable sleeve was filled with fine sand. The sleeve was then raised and locked into a position so that there was a clearance of approximately one inch between the lower end of the tube and the subgrade plate. This clearance was necessary to prevent the deflections of the bearing plate assembly from influencing the recorded subgrade deflection.

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