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Bulletin 202

Performance of Granular Subbases Under Concrete

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1958

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Performance of Granular Subbases Under Concrete

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Contents

Concrete Pavement Subbase Study in Ohio

L. D. CHILDS, Development Engineer, Portland Cement Association, and
F. E. BEHN, Assistant Research Engineer, Ohio Department of Highways

A 4-mi length of concrete surfaced arterial highway built in 1952 was designed as a test road with subbase treatment and slab length as variables. Subbases included dense-graded and open-textured crushed stone 3, 5, and 8 in. thick; cement-treated clay soil 6 in. thick, and clay soil-cement 3 and 5 in. thick. Slab lengths were 20 ft and 100 ft. Observations on performance of the road were made several times each year for five years. The periodic condition surveys were supplemented by moving load tests to observe differences in deflections and strains in the concrete slabs as related to the controlled variables.

After five years of heavy traffic, it appears that:

1. Test areas with subbases were better in over-all performance than the control areas with no subbase.
2. Open-textured limestone subbases with longitudinal edge drains prevented joint and edge pumping, and were effective in the reduction of edge blowing. Thickness of material made relatively little difference in the over-all performance of the open-textured subbases.
3. Dense-graded limestone subbases restricted joint and edge pumping and edge blowing. In this respect the 5-in. and 8-in. thicknesses were more effective than the 3-in. thickness. This was also true in the restriction of deflections under moving loads.
4. Soil-cement subbases made of clay soil and cement-modified clay soil subbases were not entirely successful because the top surface of these subbases was susceptible to erosion, which resulted in pumping at slab edges and to some extent at joints. Moving load tests, however, showed that the average record of the 5-in. soil-cement subbase, from the standpoint of restricting deflections and strains in the pavement, was equivalent to or better than that of all other subbases.
5. Little or no correlation was indicated between edge pumping and slab cracking.
6. Concrete gutters were an effective means of preventing edge pumping and blowing and greatly reduced joint pumping.

●DESIGN and construction features and preliminary results of visual observations and load tests on a 4-mi test road in Ohio have been presented previously by Allen and Childs (1). A brief review of the salient features included in the test area is given to facilitate the present discussion.

The test road is a portion of the east-bound lanes of US 20 in northern Ohio between Fremont and Clyde. This is a four-lane divided highway carrying heavy truck traffic between Toledo and Cleveland. The average week-day traffic is approximately 6,000 vehicles, of which about 2,000 are classified as heavy vehicles.

The road surface is 9-in. uniform mesh-reinforced concrete in 20-ft and 100-ft slab lengths with doweled contraction joints. The foundation consists of 12 subbase treatments built on a subgrade of clay soils of A-6(8), A-6(12) and A-7-6(13) Bureau of Public Roads classification. These materials, which are generally characterized as pumping soils, had an average of 79 percent passing a 200-mesh sieve, an average liquid limit of 35, and an average plasticity index of 16.

There were four types of subbase treatments as follows:

1. Open-textured crushed limestone having 38 percent passing a No. 4 sieve and 7 percent passing a 200-mesh sieve.

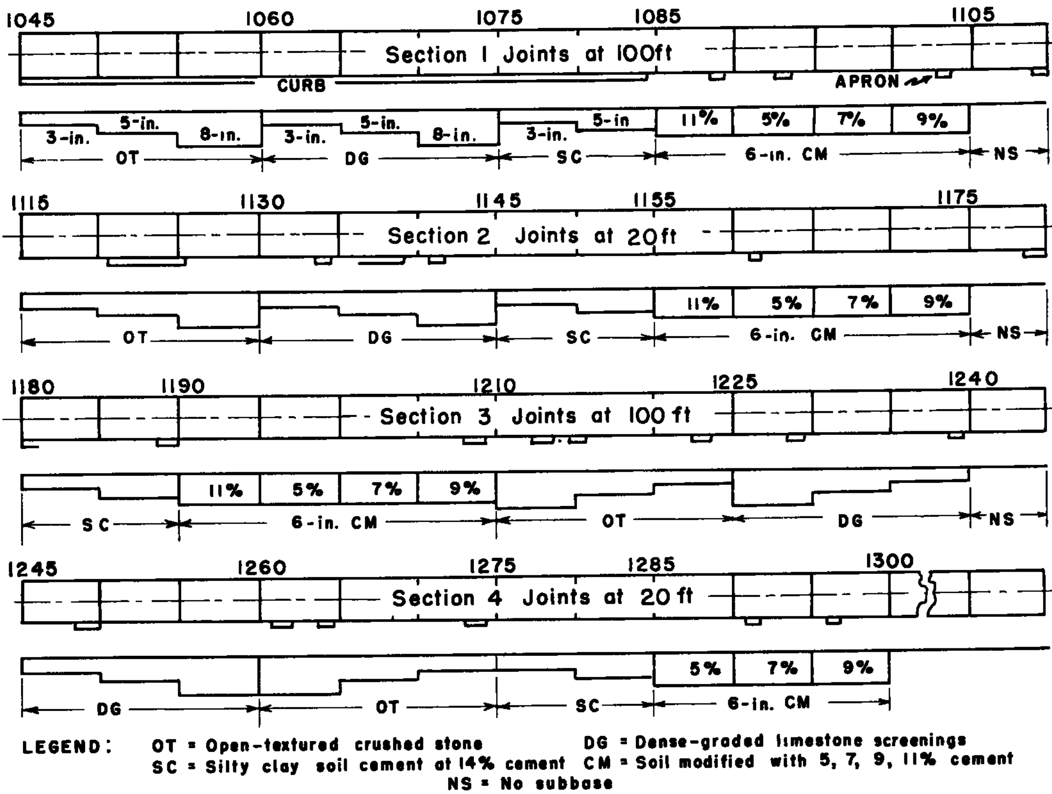


Figure 1. Location and identity of subbase treatments.

2. Dense-graded limestone screenings having 99 percent passing a No. 4 sieve and 22 percent passing a 200-mesh sieve.
3. Soil-cement made from the native clay soil with the addition of 14 percent cement.
4. Cement-modified clay soil containing lesser amounts of cement.

Figure 1 locates and identifies the subbases. Open-textured limestone was placed in 500-ft lengths at each of three thicknesses—3, 5, and 8 in. These were provided with longitudinal edge drains. They are identified hereafter as 3-OT, 5-OT and 8-OT. The dense-graded limestone screenings were also placed in three thicknesses and are identified as 3-DG, 5-DG and 8-DG. The silty clay soil with 14 percent cement met the durability and strength requirements for soil-cement, and subbases were placed in 3- and 5-in. layers. They are labeled 3-SC and 5-SC. The remaining cement-modified treatments were all 6 in. thick and contained 5, 7, 9, and 11 percent cement. They are hereafter noted as CM-5, CM-7, CM-9 and CM-11. Control sections with no subbase treatment were constructed directly on the A-6 soil and are designated NS. Figure 2 includes typical vertical sections of the road structure at a longitudinal edge of the concrete.

OBJECTIVES

The test road is cooperative project by the State of Ohio and the Bureau of Public Roads to investigate the effect of subbases on pumping control. The Portland Cement Association assisted in the tests. This is one of a series of such studies recommended by the Highway Research Board Committee on Maintenance of Concrete Pavement as Related to the Pumping Action of Slabs (3). The road provided opportunity (a) to observe the effects of subbases materials and thicknesses on pumping; (b) to study the effects of subbases on pavement strength; and (c) to study relationships between pumping and structural durability of the pavement, that is, to correlate pumping with slab deflections and stresses, and ultimately with joint faulting and slab cracking.

TRAFFIC

Traffic counts and weight surveys of vehicles on the test pavement were made periodically. The average 24-hr weekday counts for the duration of the test are given in Table 1.

Heavy vehicles are defined as all commercial trucks and buses except panel and pickup trucks. The reduction of the vehicle counts for 1956 and 1957 was undoubtedly the result of the opening in October 1955 of the parallel Ohio Turnpike.

The weight surveys indicated that for the heavy trucks carrying payloads, 60 percent

5-in. soil-cement at Sta. 1150 + 80

8-in. open-textured stone at 1127 + 00

11 percent cement-modified soil at 1159 + 20

5-in. dense-graded stone at 1135 + 40

Figure 2. Typical subbases.

TABLE 1

TRAFFIC SUMMARY

Year	Vehicles per Day	
	Heavy	Total
1953	2292	5830
1954	1969	5668
1955	2336	6036
1956	1848	4559
1957	1799	4431

of the axles carried 14,000 lb or more. However, the number of empty trucks was such that of all truck axles counted only an average of 20 percent weighed 14,000 lb or more.

The traffic data revealed that an average of 94 percent of the heavy trucks traveled on the right (outside) lane. Legal load limits in Ohio are 19,000 lb for single axles and 31,500 lb for tandem axles spaced 4 to 8 ft. Tables showing the traffic counts and weight surveys obtained since 1954 are included in the Appendix.

MOISTURE AND TEMPERATURE RECORDS

An attempt was made to measure soil moisture content electrically when the condition surveys were made. The soil moisture units installed in the subbase and subgrade were of the Colman type, a fiberglass-monel electrode sandwich moisture cell combined with a type 7A thermistor for temperature readings. Readings were taken by measuring the electrical resistance of the cells and thermistors with a self-powered alternating-current ohm-meter operating at 90 cycles per second.

Moisture cell resistance readings were reduced to approximate moisture contents by means of laboratory calibration curves. However, the moisture contents so obtained were not considered to be sufficiently accurate for this study; the cells were found to be responsive to pressure or compaction as well as moisture content. The observed readings did not in any case indicate the seasonal variations in soil moisture content which were expected.

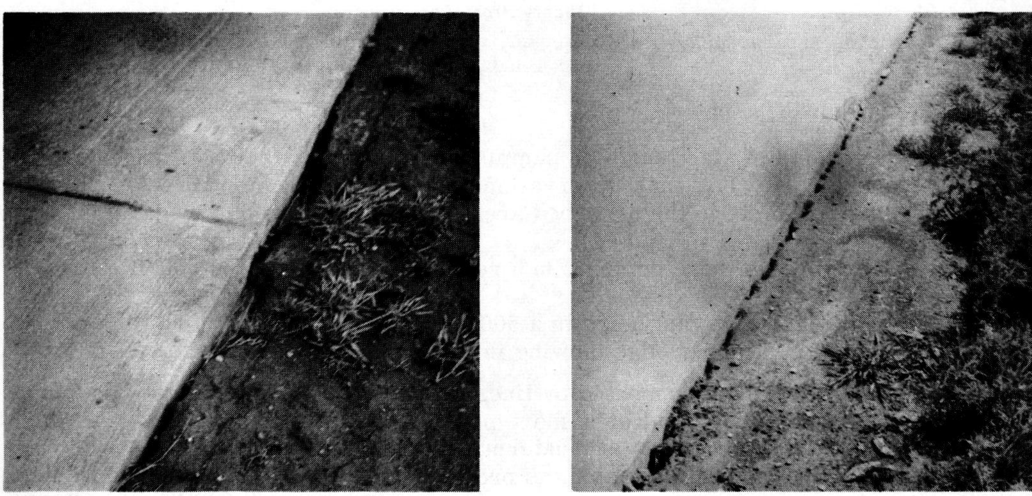
The temperature records, with the exception of a few scattered anomalous readings, did seem to follow a reasonable pattern of soil temperature distribution. On warm, sunny days the temperatures under the pavement slab were as much as 15 F higher than those at corresponding depths under sod. Isothermal lines plotted on the cross-section indicated that heat was flowing by conduction from the warm slab into the subbase and subgrade soil below. In cold weather under cloudy, overcast skies, temperatures at equal depths under pavement and under sod were similar. Isothermal lines indicated heat flow upward toward the surface.

CONDITION SURVEYS

Obsevations of pumping and blowing, begun in 1953 and described in Highway Research Board Bulletin 116 (1), were supplemented in 1954 by fault measurements at joints and by a log of the locations of cracks in the concrete slabs. All surveys and test data were obtained from observations and measurements on the righ-hand traffic lane. No edge action or joint faulting was observed in the passing lane, but a few cracks extended from the traffic lane through the center-line into the passing lane.

When the road was first opened to traffic, pumping developed on the no-subbase sections very quickly. After only two years of service, pumping joints in these sections were undersealed with bituminous material; they had already failed as controls on the subbase treatments. Observations were continued, however, and in the discussion which follows it should be noted that references to the condition of no-subbase sections since 1954 refer to pavement with undersealed joints.

Examples of pumping at joint and edge.



Blowing along edge--at joint and near mid-slab.

Figure 3. Evidence of action at slab edge.

PUMPING AND BLOWING

Visual examinations of the slab joints and edges were made in the early spring of each year and supplemented by fall and winter surveys in 1955 and 1956. It was apparent immediately that concrete gutters and aprons inhibited the edge and joint action. Although records were maintained on these areas, the analysis attempted here is based on uncurbed lengths of pavement where edge and joint action were readily observable. Summaries of the number of pumping joints, occurrences of edge pumping, occurrences of edge blows, and lineal feet of continuous blowing—all based on 100 ft of uncurbed slab length are given in the Appendix. Some of the conditions previously described are shown in Figure 3.

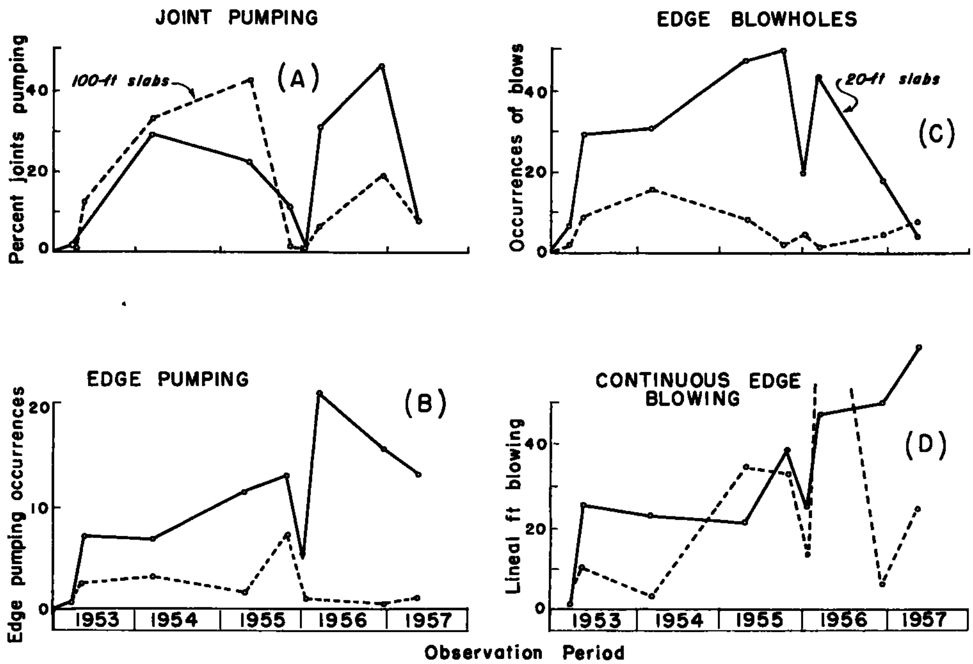


Figure 4. Pumping and blowing vs slab length, grand average of all treatments; 500-ft basic length.

Influence of Slab Length

To study the effects of slab length on pumping and blowing, the four graphs of Figure 4 were constructed. For each observation period the following data were plotted:

1. Percentage of joints in the uncurbed areas which were pumping regardless of severity.
2. The occurrences of edge pumping in a representative uncurbed 500-ft length of road.
3. The instances of edge blowholes in a 500-ft length.
4. The length of continuous edge blowing in lineal feet for an average 500-ft length.

In Figure 4 the points are connected by lines for the purpose of identity. Instances of pumping are greatly reduced after a short period of dry weather, so the behavior of the graph between plotted points is indeterminable.

Figure 4A suggests that joint pumping is probably independent of slab length. When observing the phenomenon on the road one is apt to conclude that joints between 20-ft slabs are more susceptible to pumping than joints between 100-ft slabs. However, there are five times as many joints in a length of 20-ft slabs as in a length of 100-ft slabs and the observer is influenced by quantity.

Edge pumping was more severe along edges of short slabs than on long ones. This is demonstrated by Figure 4B where the edge pumping occurrences for 20-ft slabs are several times greater than for 100-ft slabs. Also, there is an indication that edge pumping tends to become greater with time in areas with short slabs and to decrease at the edges of long slabs.

Edge blowholes were more frequent along 20-ft slabs than along 100-ft slabs. This is corroborated by Figure 4C with the exception of the 1957 observations.

Continuous edge blowing seemed to increase with time, especially in sections with 20-ft slabs. The severity of this phenomenon along 100-ft slabs was great in the spring of 1956; but aside from that and a small discrepancy in 1955, continuous edge blowing was worse along the edges of short slabs than along the 100-ft slabs.

Effect of Subbase Treatment

Joint Pumping. The percentage of pumping joints in each treatment based on the total number of pumping joints observed is given in Figure 5. These data are from the critical spring readings for each of the five years of observation. The following conclusions appear to be warranted:

1. Open-textured crushed stone was most effective in the prevention of joint pumping. In this respect the 3-in. thickness was as good as the 5-in. and 8-in. thicknesses.
2. Dense-graded, although not as good as open-textured stone, was still a good material for inhibiting pumping. Thickness of dense-graded material seemed to be a factor in its performance. In the early years some joint pumping was seen among 20-ft slabs on the 8-in. treatment, but this diminished and in 1956 and 1957 joint pumping in these areas was found on the 3-in. thicknesses. In 1955, 15 to 19 percent of the joints between 100-ft panels on 3-in. dense-graded stone pumped; in both 1955 and 1956 there was pumping at joints in the 5-in. thickness. Of the dense-graded treatments in this road, the 8-in. thickness appeared to be best; the 5-in. was slightly better than the 3-in. thickness.
3. The silty clay soil-cement of this project was not successful in preventing joint pumping. Erosion of the top surface of this material apparently developed due to a combination of water between the slab and the subbase and pavement deflections under moving truck loads. This slurry was ejected at joints and slab edges. In most cases there were more pumping joints on the 3-in. than on the 5-in. thickness. The percentage of pumping joints between 100-ft slabs was always greater than the percentage of pumping between 20-ft slabs when silty clay soil-cement was the subbase. In the 100-ft slab areas, joint pumping over the 3-in. soil-cement treatment was sometimes more and sometimes less than that in areas with no subbase, but with one exception—there was always less joint pumping over 5-in. soil-cement than over the control areas with no subbase.
4. The remaining cement-treated subbases were of some benefit in the reduction of joint pumping below that of the no-subbase area, but they did not prevent joint pump-

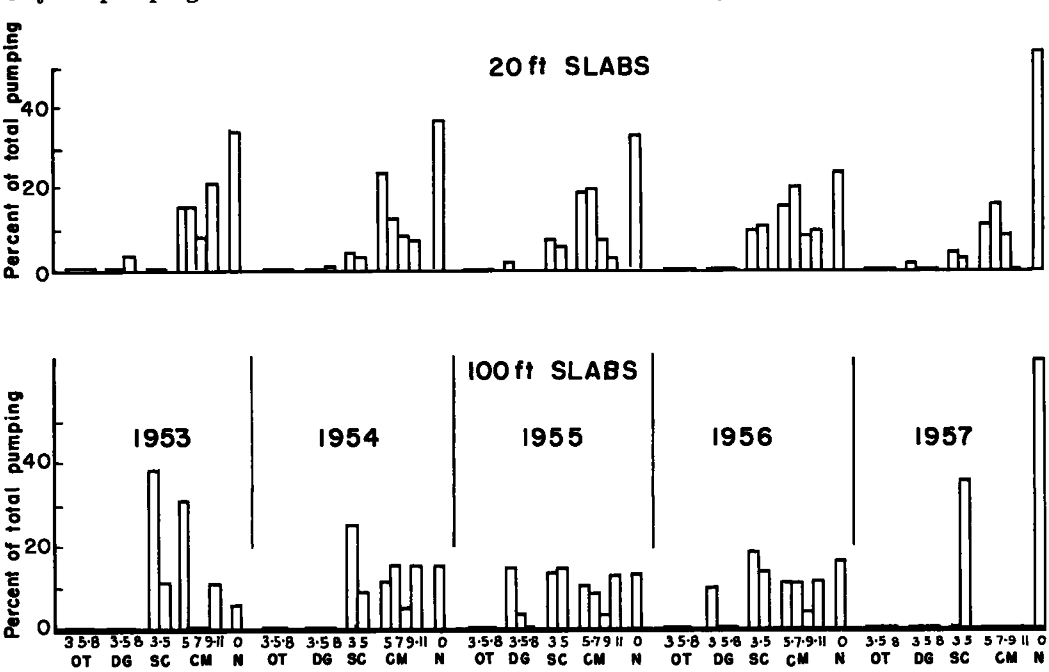


Figure 5. Joint pumping.

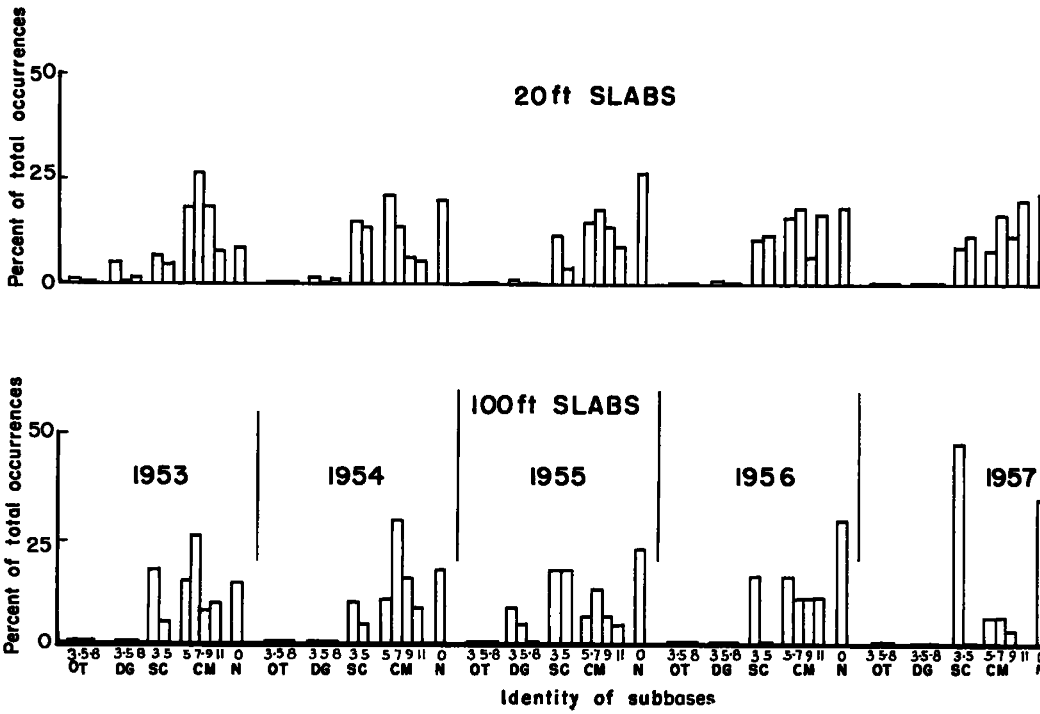


Figure 6. Edge pumping.

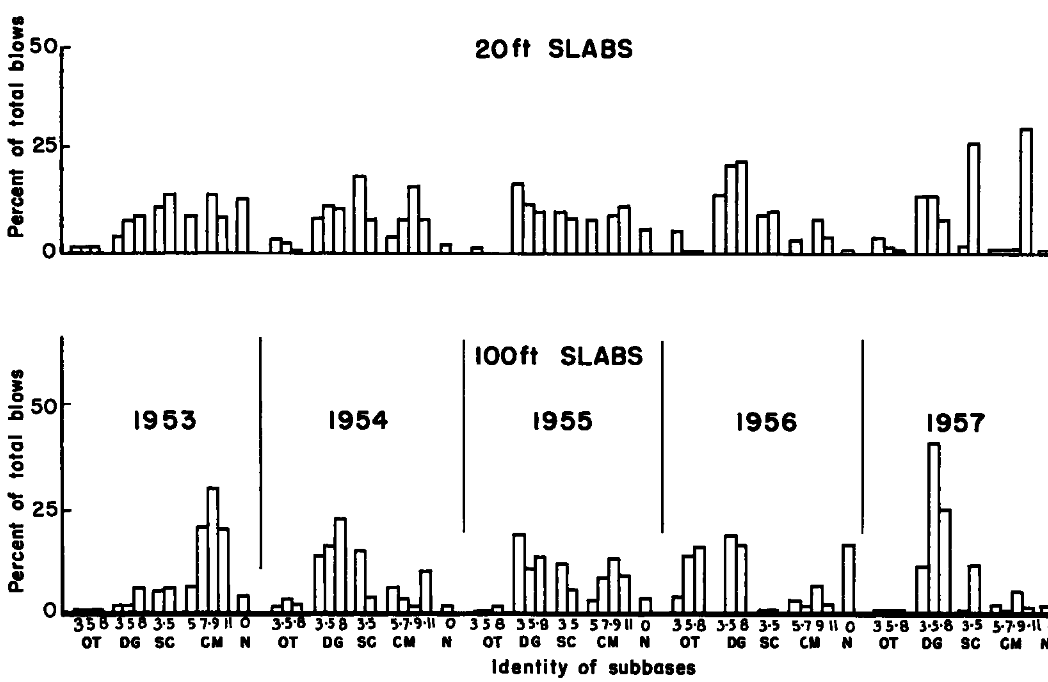


Figure 7. Edge blowing.

ing. Among 20-ft slabs there was less joint pumping over the cement-modified treatments than in the areas of no subbase. However, this was not always the case when the slab length was 100 ft. The 9 percent cement-treated subbase was generally the most effective of the cement-modified treatments in the reduction of joint pumping. In most cases, the number of pumping joints in this treatment was from $^1/_4$ to $^1/_3$ of the number found in control areas without subbase.

5. It is difficult to classify CM-5, CM-7, and CM-11 as to their superiority in joint pumping reduction. CM-11 appears to be slightly better under 20-ft slabs, but loses its advantage under 100-ft slabs. CM-5 and CM-7 were for the most part about equally effective.

In summary, the observations indicate, with minor exceptions, that all treatments were of some benefit in the reduction of joint pumping below that in untreated areas. All open-textured treatments were successful in preventing joint pumping and there is no preferential thickness, the 3-in. thickness being equally as effective as the 5- and 8-in. thicknesses. Next in ability to reduce joint pumping are 8-DG, 5-DG, and 3-DG in that order. Of the cement treatments, 5-SC is next, followed by CM-9 and 3-SC. CM-11 showed slight advantage over CM-5 and CM-7.

Edge Pumping. Relationships between edge pumping and subbase treatment were similar to those between joint pumping and subbase treatment. Figure 6 gives the percentage of edge pumping in each treated area based on the total observed occurrences of edge pumping. The open-textured subbases with drains prevented edge pumping almost entirely; the dense-graded subbases were effective, but not as successful in edge pumping prevention as open-textured treatments. The effect of thickness on the performance of dense-graded subbases with respect to edge pumping was minor, with the 8-in. showing slight superiority over the 5-in. thickness, which in turn was a little better than the 3-in. thickness.

Of the cement-treated subbases, 5-SC appeared to permit less pumping than the remaining treatments. The CM-11 and CM-9 treatments permitted less edge pumping than the control except in one instance, and rated better than CM-5 and 3-SC. CM-7

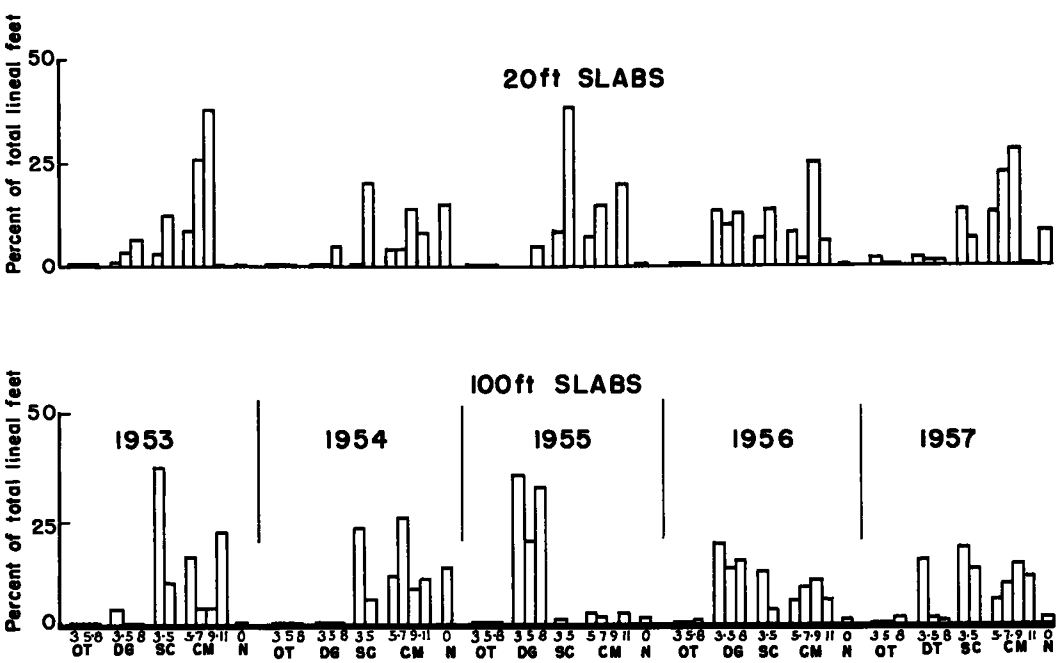


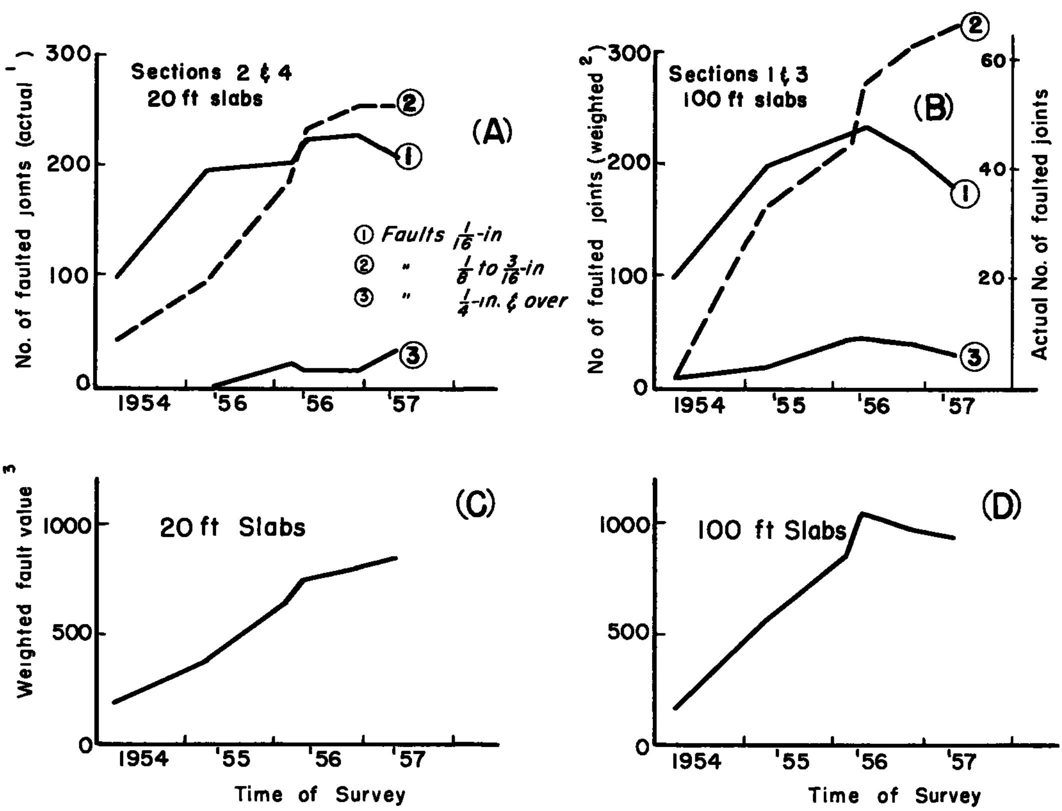
Figure 8. Continuous blowing.

was less effective in the inhibition of edge pumping than other treatments, but on the average was a small improvement over no subbase.

Edge Blowing. Figure 7 relates subbase treatment to the percentage of edge blows observed in that area. It is evident that edge blowing prevailed in all areas at some time. The open-textured material was effective in holding the occurrences of edge blows to a minimum, but it did not prevent blowing. The thickness of open-textured subbase did not appear to be significant.

All remaining subbase treatments were ineffective in the reduction of edge blowing. In fact, in almost all cases there was less edge blowing along slabs built directly on the subgrade than along slabs with dense-graded or cement-treated subbase.

Continuous Blowing. Except for the case of open-textured subbases, Figure 8 presents a somewhat confused picture of the relations between continuous blowing and subbase treatment. Second to open-textured crushed stone in effectiveness in the prevention of continuous blowing is the control area with no subbase. Some blowing was seen in the control areas in 1954 and a little in 1957, but on the average it was less than that observed in the dense-graded and cement-modified treatments. Probably dense-graded treatments should be rated third in effectiveness in spite of the high percentage of continuous blowing along 100-ft slabs in 1955 and on the 3-in. thickness in 1956 and 1957. The soil-cement and cement-modified treatments ranged over great extremes and they are difficult to classify. In any case, they are less effective in the prevention of continuous blowing than no subbase.



1 Actual values doubled for years '54 and '55 (see text)

2 Weight factor 5 = ratio of No. of joints in Sections 2 & 4 to No. in Sections 1 & 3

3 Class ① plus twice class ② plus four times class ③

Figure 9. Development of faulting.

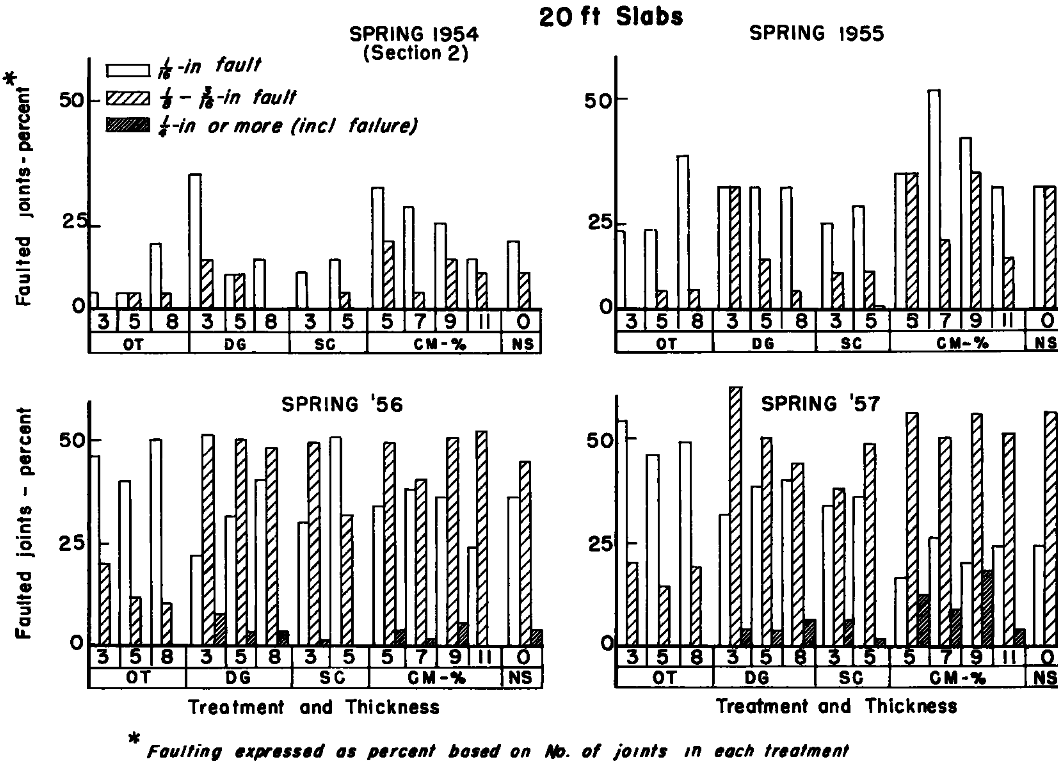


Figure 10. Joint faulting.

JOINT FAULTING

In the spring of 1954, measurements of the faulting which developed at joints were begun. The change in elevations of the forward slab in the direction of traffic with respect to the adjacent slab immediately behind it was recorded in increments of $^1/_{16}$ in. and the direction of fault was noted. For the analysis the data were grouped into Class 1($^1/_{16}$ in.), Class 2 ($^1/_8$ to $^3/_{16}$ in.), and Class 3 ($^1/_4$ in. and greater) faults. This classification system does not include the case where the forward slab is higher than the reference slab.

A summary of classified fault data is given in the Appendix. In 1954 and 1955 the measurements were restricted to sections 2 and 3, which constitued one-half the project and consequently provided data on one-half the total number of joints. The data for subsequent years comprised all joints in the project, including those in the curbed sections. A graphical portrayal of faulting development is shown in Figure 9, where the 1954 and 1955 data are weighted by a factor of 2 to compensate for the shorter test section. Also, for comparison of slab length effect, the data for 100-ft slabs were weighted by 5 because there were only one-fifth as many joints in the sections with 100-ft slabs as in sections with 20-ft slabs. Curves (C) and (D) of Figure 9 are constructed on the premise that severity of faulting is proportional to the size of the fault. Thus the weighted fault value, plotted as the ordinate, is the sum of Class 1 faults plus twice the Class 2 faults plus four times the Class 3 faults.

Influence of Slab Length

There is reasonable similarity in trend in the development of Class 1 and Class 2 faulting in both 20-ft and 100-ft sections. The weighted number of Class 2 faults is larger for the long slabs. In 1956 and 1957 Class 1 faults on long slabs decreased more rapidly than Class 1 faults on short slabs. This suggests an upgrading in severity of faulting. The decrease in Class 3 faults in 100-ft slabs in 1956 and 1957

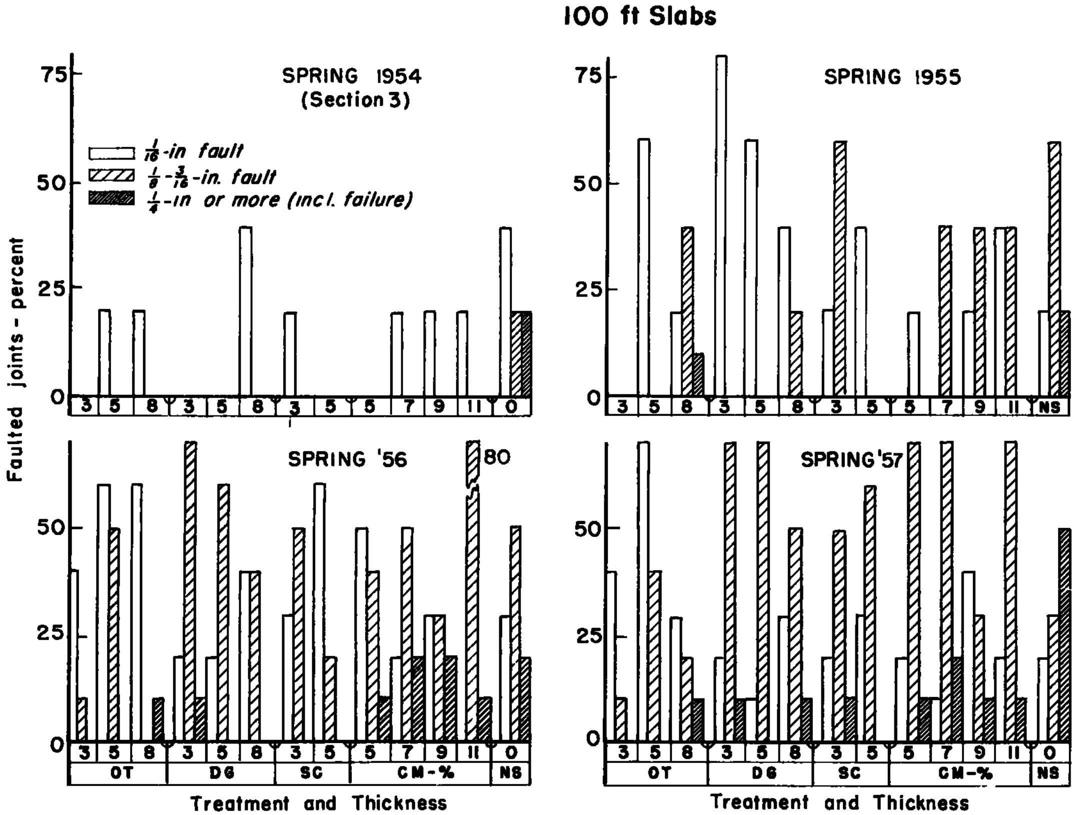


Figure 11. Joint faulting.

may have been due in part to the formation of transverse cracks within 10 to 12 ft of slab ends. The resulting stress relief permitted reorientation of the slab end with respect to subgrade and may have decreased the fault classification from Class 3 to Class 2.

The weighted fault value curves C and D of Figure 9 indicate greater fault severity between 100-ft slabs than between 20-ft slabs. In May 1956 the fault value for 100-ft slabs was one-third greater than that for 20-ft slabs, but it was only one-eighth greater in 1957.

Influence of Subbase

The joint faulting data for the critical spring surveys of 1954, 1955, 1956 and 1957 were classified by severity and separated by slab length and subbase treatment to obtain Figures 10 and 11. The ordinates for these charts are the percent of faulted joints based upon the number of joints in each category. There were 50 joints over each subbase treatment with 20-ft slabs and 10 joints for 100-ft slabs, with the exception that Section 4 omitted the CM-11 and NS areas leaving 25 joints for each of these treatments with 20-ft slabs. In 1954 only one-half the project was surveyed.

Faulting began to appear to a degree which complicated the analysis of subbase influence in the spring of 1956. The general distribution of faulting throughout the test road suggested a weakness in joint construction. This was corroborated by observations of general joint spalling.

In 1957 cores were taken from the pavement at a few joints. They were drilled at locations 30 in. inward from the free edge to include one of the dowels. Elongation of the dowel socket in the approach slab was noted in some instances and honeycombing was found under some of the dowels. This lack of dowel support may have contributed in some degree to the magnitude of faulting. The degree to which these conditions

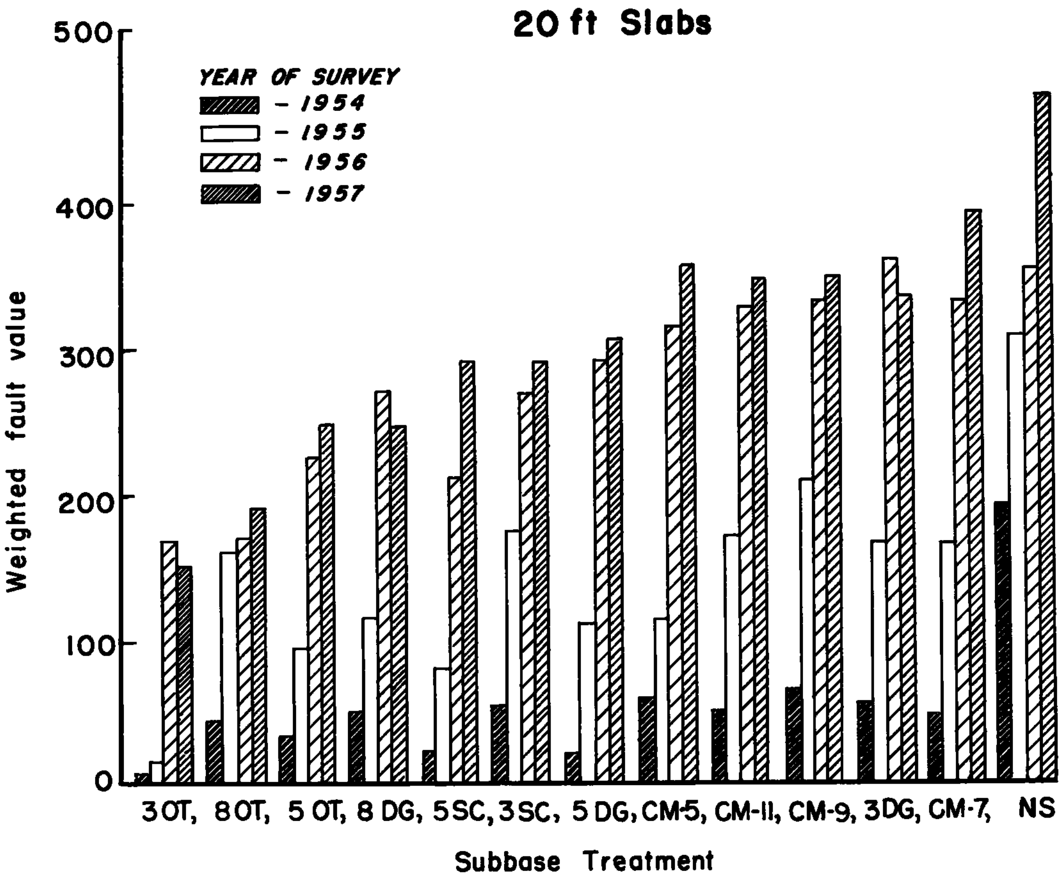


Figure 12. Effect of subbase on faulting.

increased the fault cannot be determined and for this analysis it is assumed that all joints are affected equally.

Figures 10 and 11 are too complicated to permit satisfactory analysis by inspection, but a few general remarks may be in order. Slabs on open-textured subbases developed less Class 2 faults than on other subbases in the 20-ft slab sections and there were no Class 3 faults. In sections with 100-ft slabs the faulting in areas with open-textured subbases was more severe than in similar areas with 20-ft slabs, but open-textured subbases were still among the treatments with least faulting. Soil-cement appeared to be slightly better in fault resistance than dense-graded material.

On the basis of weighted fault value, it is possible to rank the subbases in order of their resistance to faulting. Figure 12 is drawn with emphasis on the weighted fault values for the last two years. Comparisons are made with the reservation that the faulting on the project is not representative of standard pavement construction.

It is seen that the faulting was least in areas with 3-in. and 8-in. open-textured subbases. The 5-OT was slightly better than 8-DG. The 5-SC, 3-SC and 5-DG were the next group, followed by all of the cement-modified treatments and 3-DG. Most susceptible to faulting was the control area without subbase.

DEVELOPMENT OF TRANSVERSE CRACKS

To complete the condition survey of the road, a map was drawn showing the locations of all of the full-depth transverse cracks which developed during the five years of observation. From these maps the number of cracks were tallied and the data for the spring observations of 1954, 1955, 1956 and 1957 are shown graphically in Figure 13.

Transverse cracks were infrequent during the early years, but between March 1956 and May 1957 a large number of cracks developed. The survey of December 1956 showed that many of these had formed prior to the December survey, but at least as many more developed between December and the following May.

The record exhibited in Figure 13 is for the right-hand traffic lane. No data are shown for the passing lane, but the cracking in that lane was very limited and almost entirely confined to extension of cracks in the right-hand lane in long slabs at distances well away from the joints.

Effect of Slab Length

Some transverse cracking in 100-ft slabs is expected and the design of the pavement takes this into consideration. Cracking, however, is not expected in 20-ft slabs. Ac-

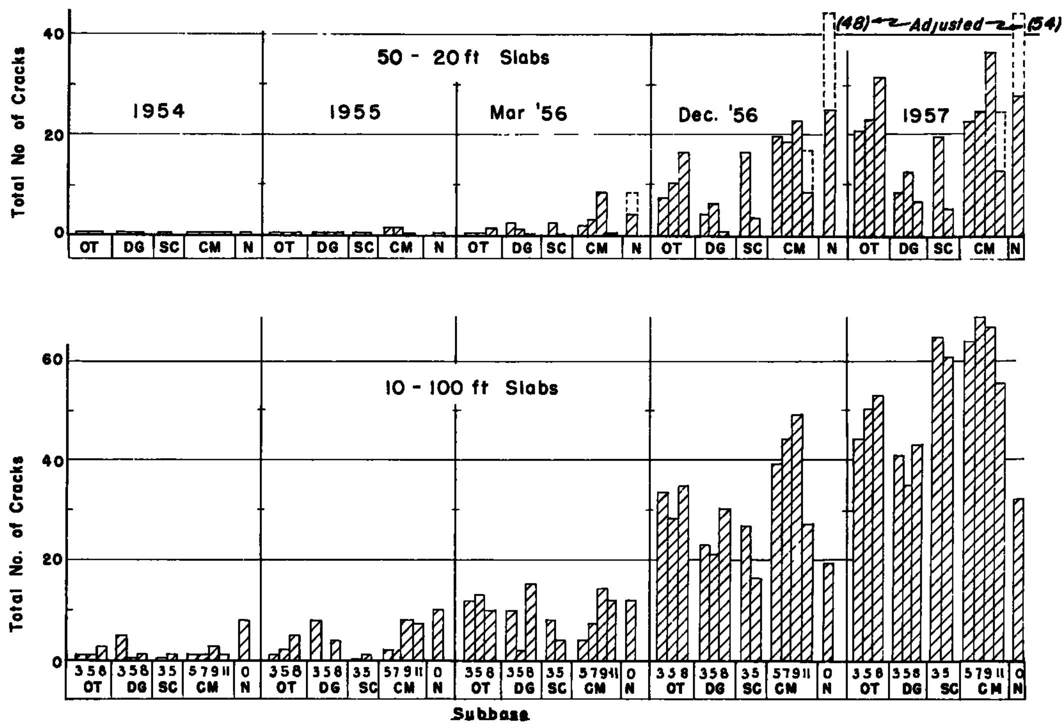


Figure 13. Transverse cracks.

cordingly, cracks that occur in 20-ft slabs are generally considered more serious than those that occur in 100-ft slabs.

Figure 13 shows more cracks in 100-ft slabs than in 20-ft slabs. However, for one type of analysis, a contraction joint in a pavement may be considered a controlled crack, and if the cracks through the joints are added to the cracks observed, and the long-slab and short-slab sections are compared on a basis of total cracks per 100 ft, it is seen that there are more cracks in sections with 20-ft slabs than in sections with 100-ft slabs. This observation is substantiated in Table 2, which is a summary of the crack survey data of May 1957.

Influence of Subbase

From Table 2 it is apparent that areas of least cracking under 20-ft slabs do not necessarily have the same subbase as areas of least cracking under 100-ft slabs. For example, in the 20-ft slab sections, where complete elimination of all cracking is desirable, 5-SC, 8-DG and 3-DG had less than 1 crack per 100 ft, and 5-DG followed closely with 1.2 cracks per 100 ft. In the next group were 3-SC, 3-OT, 5-OT, CM-5,

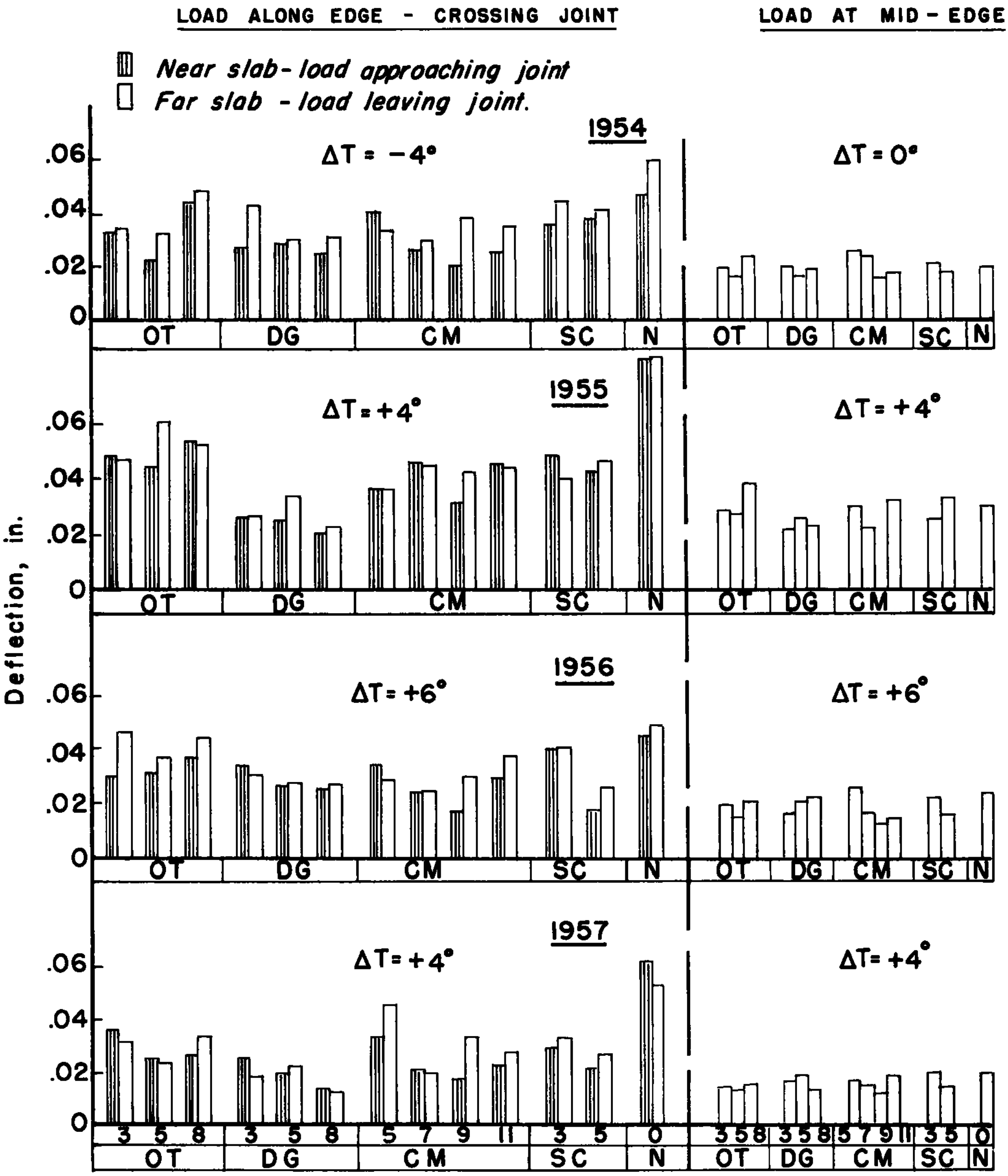


Figure 14. Early morning deflections; 31,500 lb on tandem axles, 20-ft slabs.

TABLE 2

TRANSVERSE CRACKS IN RIGHT-HAND TEST LANE

Slab Length (ft)	Cracks per 100 Ft in Designated Treatment: 3OT	5OT	8OT	3DG	5DG	8DG	3SC	5SC	CM5	CM7	CM9	CM11	NS
20	2.0	2.2	3.3	0.8	1.2	0.6	1.9	0.5	2.2	2.4	3.6	2.4	5.4
100	4.4	5.0	5.3	4.1	3.5	4.3	6.4	6.0	6.3	6.8	6.6	5.5	3.2
20[1]	6.0	6.2	7.3	4.8	5.2	4.6	5.9	4.5	6.2	6.4	7.6	6.4	9.4
Excess[2]	1.6	1.0	2.0	0.7	1.7	0.3	-0.5	-1.5	-0.1	-0.4	1.0	0.9	5.8

[1] Adjusted value = 20-ft count plus 4 contraction joints.
[2] 20-ft adjusted less 100-ft observed.

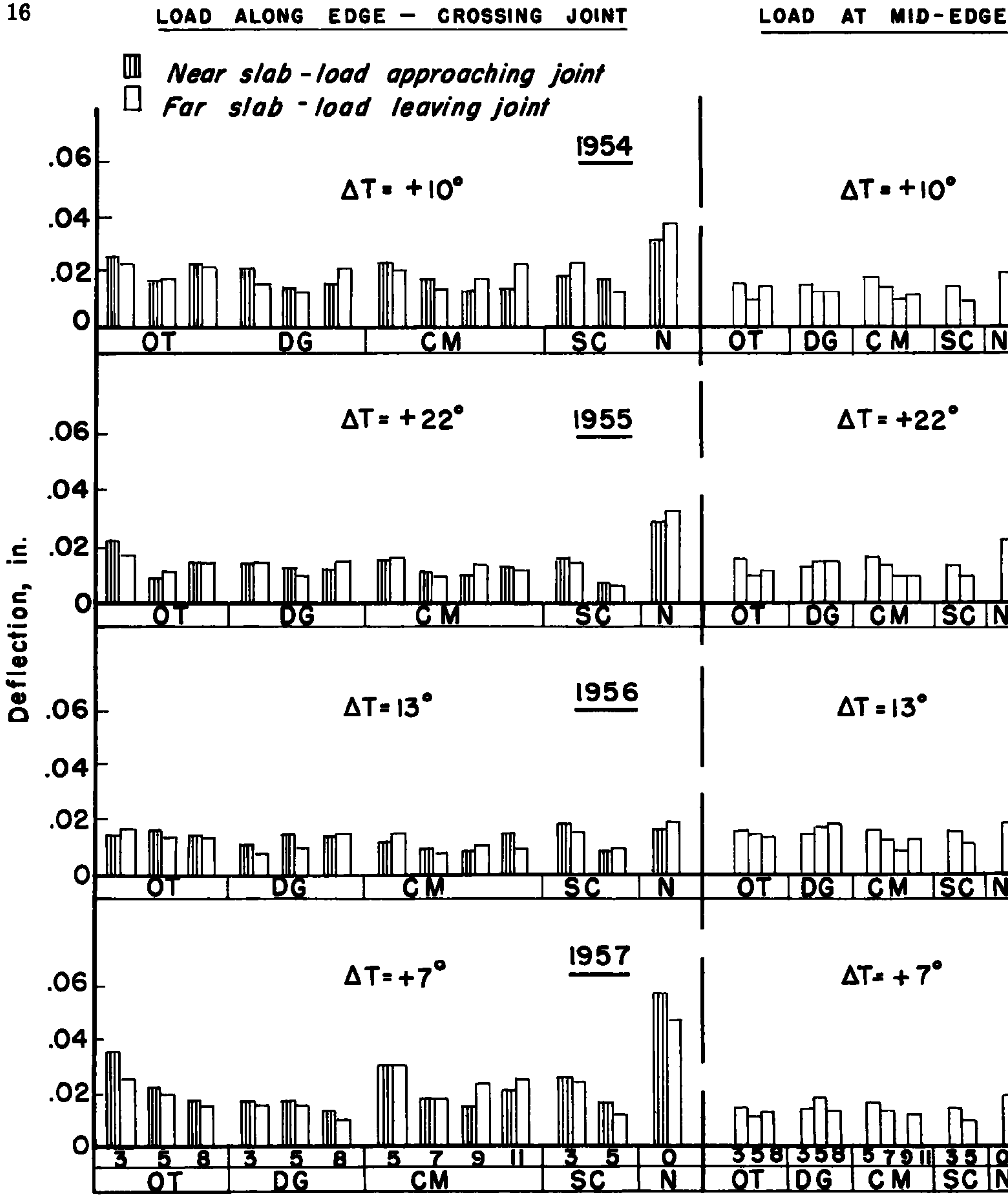


Figure 15. Afternoon deflections; 31,500 lb on tandem axles, 20-ft slabs.

CM-7 and CM-11 with 1.9 to 2.4 cracks per 100 ft; 8-OT and CM-9 had more than three cracks per 100 ft, and there were 5.4 cracks in 100 ft of slabs without subbase treatment.

In 100-ft slab sections the control length without subbase had least cracks and 5-DG was next with 3.5 cracks per 100 ft. Treatments 3-DG, 8-DG and 3-OT had 4.0 to 4.4 cracks per 100 ft. Next were 5-OT, 8-OT and CM-11 with 5.0 to 5.5 cracks per 100 ft, followed by 5-SC, CM-5, 3-SC, CM-9, and CM-11, which had 6.0 or more cracks per 100 ft.

The only subbases which rate well under both slab lengths are the three dense-graded thicknesses; 3-OT and 5-OT were fair in crack resistance, but 8-OT was poor under 20-ft slabs. 5-SC was best under 20-ft slabs, but was well down the line under 100-ft slabs. Likewise the 100-ft slabs with no subbase had the least cracking, but 20-ft

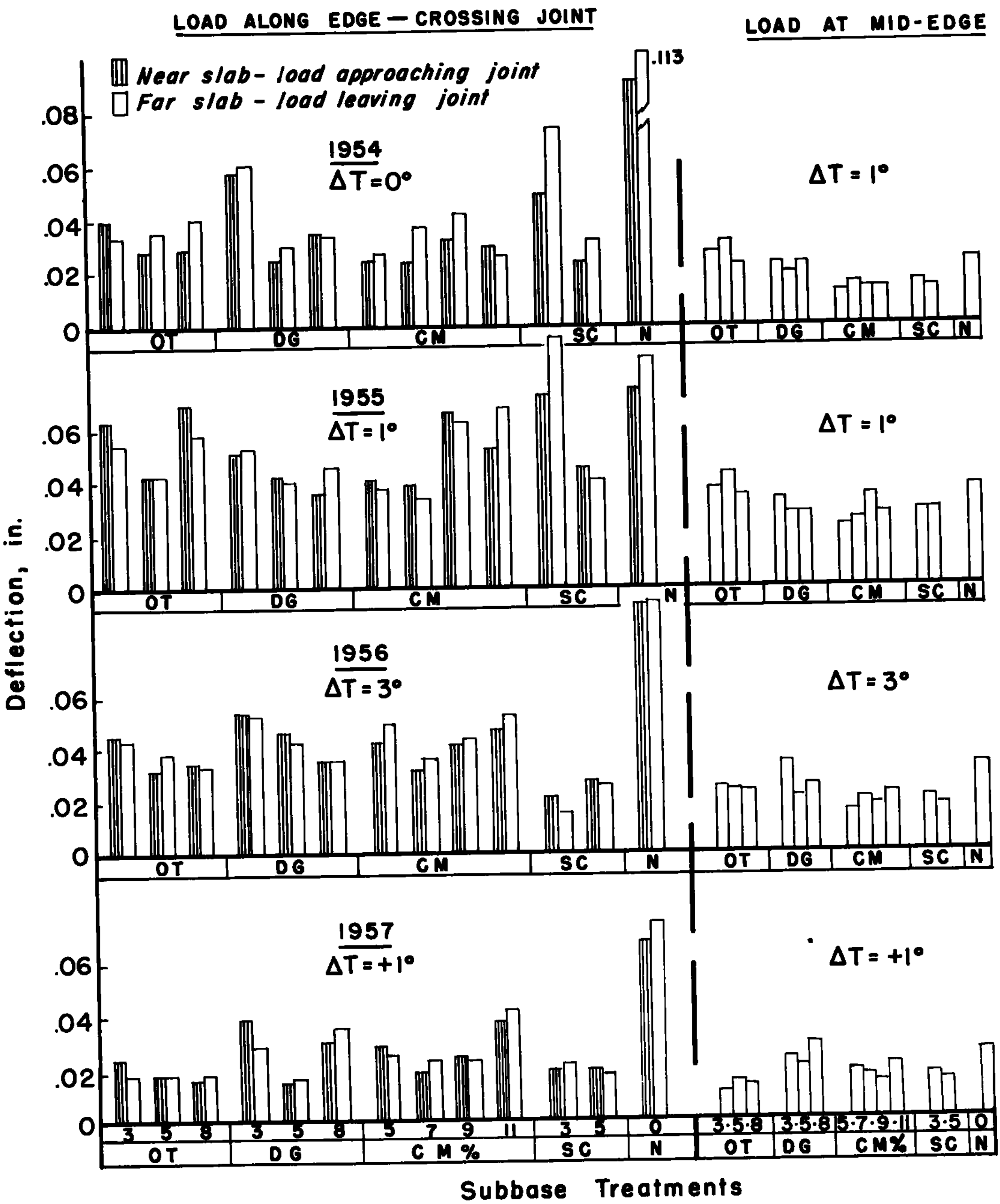


Figure 16. Early morning deflections; 31,500 lb on tandem axles, 100-ft slabs.

slabs without subbase had the most. The CM-9 modification was poor under both slab lengths.

DEFLECTIONS AND STRAINS

The two types of deflection measurement described in HRB Bulletin 116 (1) were repeated throughout the five-year period. The first was a measurement of maximum slab deflections at edges and corners under moving controlled loads in representative areas of each subbase for a full section; the second was a record of the complete deflection curves at edges and corners for selected areas in several treatments. Measurements

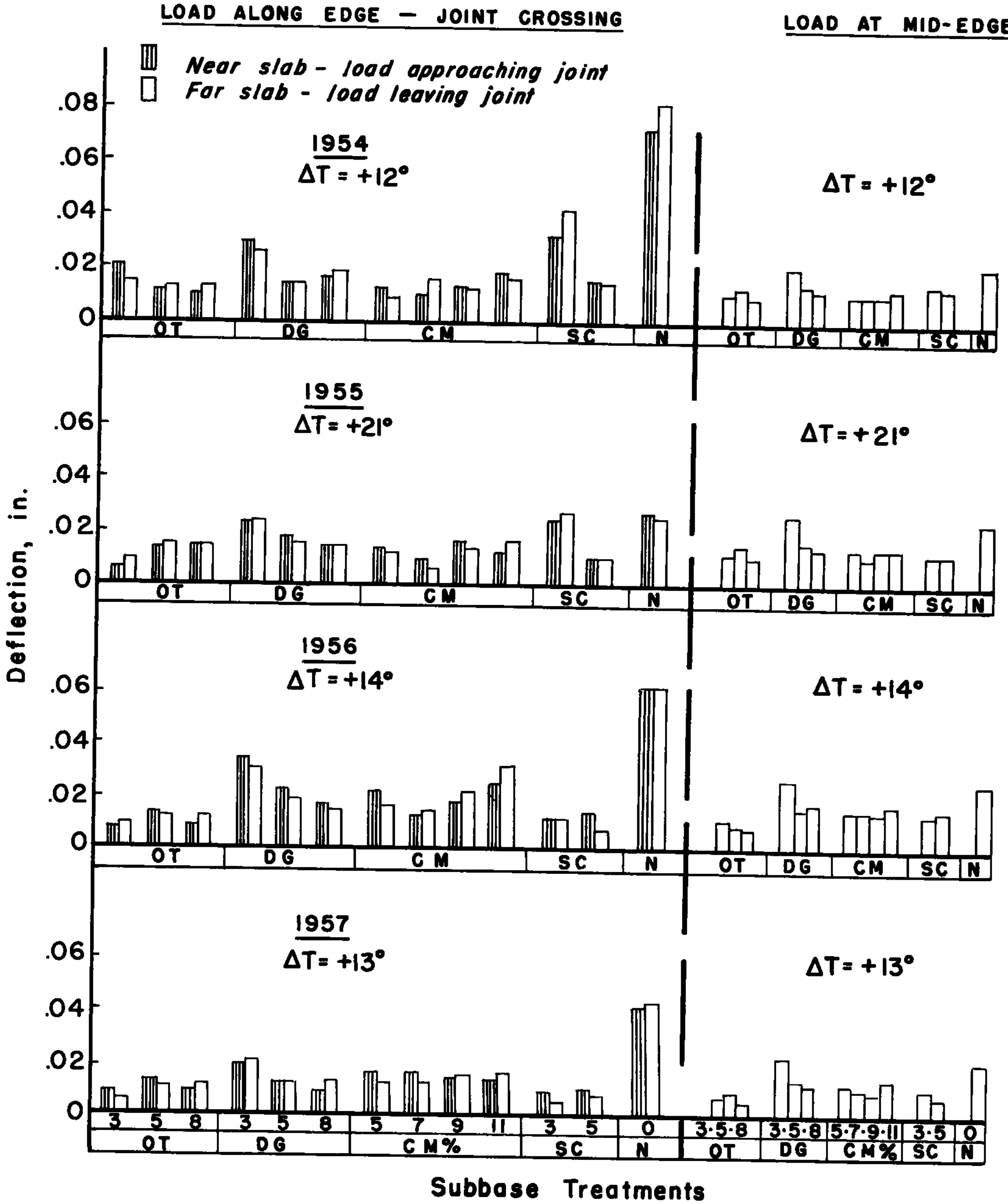


Figure 17. Afternoon deflections; 31,500 lb on tandem axles, 100-ft slabs.

of critical surface strains at slab edges were made simultaneously with the dynamic deflection records.

MAXIMUM DEFLECTION STUDY

The deflectometer for the measurement of maximum deflections utilized a steel pin held in a machined brass housing by a leather friction pad; the housing was attached to the vertical face of the concrete slab by a bracket and the steel pin rested upon a rod driven through a casing deep into the subgrade. The length of pin extending above the housing was read with a 0.001-in. dial indicator equipped with an adapter. The change in this length occurring as the housing was forced to slide down over the pin by passage of the test vehicle wheels along the slab edge was recorded as the maximum deflection.

The test vehicle for this operation was a 3-axle dump truck with a 31,500-lb load on the rear tandem axles. The driver became proficient at steering his truck with the outside walls of the rear tires within 2 or 3 in. of the slab edge. The test speed was about 5 mph.

To minimize the effect of temperature variations during the test run, the time was made very short. Five engineers were each assigned 15 deflectometers and each man was able to read deflections in his assigned section, reset the pins and read new zeros in the 20 min required for the truck to return on the next run. Three runs were made for each test. The same area was tested in early morning and mid-afternoon to observe the effects of temperature differential and resulting slab curl on the maximum deflections. Figures 14 through 17 have been prepared from the tabulated data given in the Appendix. Because the tests in 1953 were made in summer instead of spring, the deflections and strains were affected by lower subbase moisture and higher slab temperatures, and these data are not included in the figures.

TABLE 3

EFFECT OF TEMPERATURE ON DEFLECTION;
Mean Deflection, d (0.001 in.), at Indicated Temperature Differential, T (F)

Time	1954				1955				1956				1957			
	Corner		Edge		Corner		Edge		Corner		Edge		Corner		Edge	
	T	d	T	d	T	d	T	d	T	d	T	d	T	d	T	d
(a) 20-ft slabs																
A. M.	-4	36	0	20	4	44	4	27	6	32	6	18	4	29	4	16
P. M.	10	21	22	13	13	13	7	23	10	14	22	12	13	14	7	14
Diff.	14	15	22	7	9	31	3	4	4	18	18	6	9	15	3	2
d/T[1]	1.1		0.3		3.4		1.3		4.5		0.4		1.7		0.7	
(b) 100-ft slabs																
A. M.	0	43	1	21	-1	56	-1	32	3	43	3	23	1	28	1	19
P. M.	12	23	12	14	21	15	21	15	14	20	14	16	13	16	13	17
Diff.	12	20	11	7	22	41	22	17	11	23	11	7	12	12	12	2
d/T	1.7		0.6		1.9		0.8		2.1		0.6		1.0		0.2	

[1] d/T=Change in deflection per degree temperature difference between top and bottom of slab.

Influence of Temperature and Slab Length

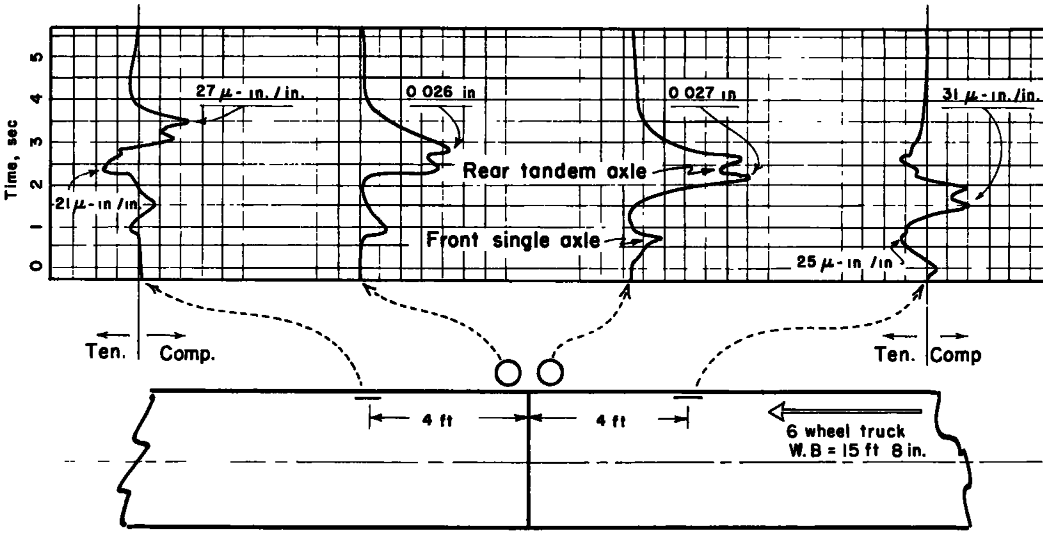
Figure 14 may be compared with Figure 15 and Figure 16 with Figure 17 to observe the increase in deflection resulting from low-temperature differential between top and bottom of the concrete. The magnitudes of these differences are more clearly shown by the comparison of means of deflection in Table 3.

The doweled corners of the pavement slabs were three to four times as sensitive to changes in temperature differential as were the edges. For 20-ft slabs the grand average of the d/T values for corners is 0.0027 and for edges it is 0.0007 in. For corners of 100-ft slabs d/T averages 0.0017 as compared to 0.0006 in. for edges. These values indicate that the 20-ft slabs were more responsive to temperature differential than were the 100-ft slabs.

Influence of Subbase Type

A brief inspection of Figures 14 to 17 is sufficient to warrant the statement that subbases in general were of benefit in the reduction of slab deflections. The advantage of subbases was not as great at the pavement edges as at the corners, but the figures show that with very few exceptions the roadway structure benefited greatly under corner loading; that is, when the truck, with outer wheels on the slab edge, traveled across the joint.

Morning tests on 20-ft slabs indicated that 8-DG consistently reduced corner deflections to 50 percent or less of control area deflections. All dense-graded subbases were generally more effective than open-textured subbases. Cement-treated subbases



Sta. 1216 + 50, 5-OT, 100-ft slab

Figure 18. Deflections and strains at joint; 31,500-lb tandems moving along slab edge.

were slightly better than open-textured subbases, but not as good in deflection reduction as the dense-graded areas. At slab edges, CM-9 reduced deflections more consistently than other cement treatment.

Afternoon tests on 20-ft slabs showed corner deflections to be least on subbases 5-SC, 5-DG and CM-9. The other two dense-graded treatments and CM-7 were next in effectiveness, followed by 5-OT and 8-OT. At edges, CM-9 was most effective.

Morning tests on 100-ft slabs showed that corner deflections were least on subbases of 5-DG, 5-SC, 5-OT and CM-7. In 1957 all subbases reduced deflections 50 percent or more below those in the control area. At this time CM-11, 3-DG and 8-DG were not as effective as the others. At slab edges, deflections were lower over cement-treated subbases than over the granular treatment, except in 1957 when the open-textured treatments were most effective.

Afternoon records of tests on 100-ft slabs showed that all treatments except 3-DG and 3-SC were very effective in reducing corner deflections. In 1957 all treatments were good in this respect. At edges, 3-DG allowed deflections as large as those on the control area, but all other treatments reduced deflections by at least 30 percent. In 1957 the most effective were the three open-textured subbases and CM-9 and 5-SC.

In summary, with few exceptions all subbase treatments reduced slab deflections appreciably below those measured in the control areas. Although no single treatment or group was consistently best in all tests, it appears that deflections were reduced most consistently by treatments 5-SC, 5-DG, 8-DG, and CM-9. Of the open-textured treatments the 3-in. and 5-in. thicknesses were more effective than the 8-in. thickness. This trend was not substantiated in the dense-graded series, where 3-DG was the least effective. CM-5, CM-7, CM-11, and SC-3 were among the poorer subbases with respect to deflection reductions, although they were considerably superior to control.

MOVING LOAD STUDY

The scope of the study of strains in the concrete slabs due to applied loads was limited because strain measurements required electronic instrumentation. Strains on the slab upper surface at the outside edge at mid-slab and near corners were measured at treatments 5-OT, 5-DG, 5-SC, CM-7, and NS. This was accomplished by the use of SR-4 type A-9 gages and a four-channel automatic recorder.

Exploratory tests were made to find the distances from the joint at which maximum strain was developed when the load crossed the joint. These distances were used as the spacing for the SR-4 gages throughout the section tested. Other gages were cemented

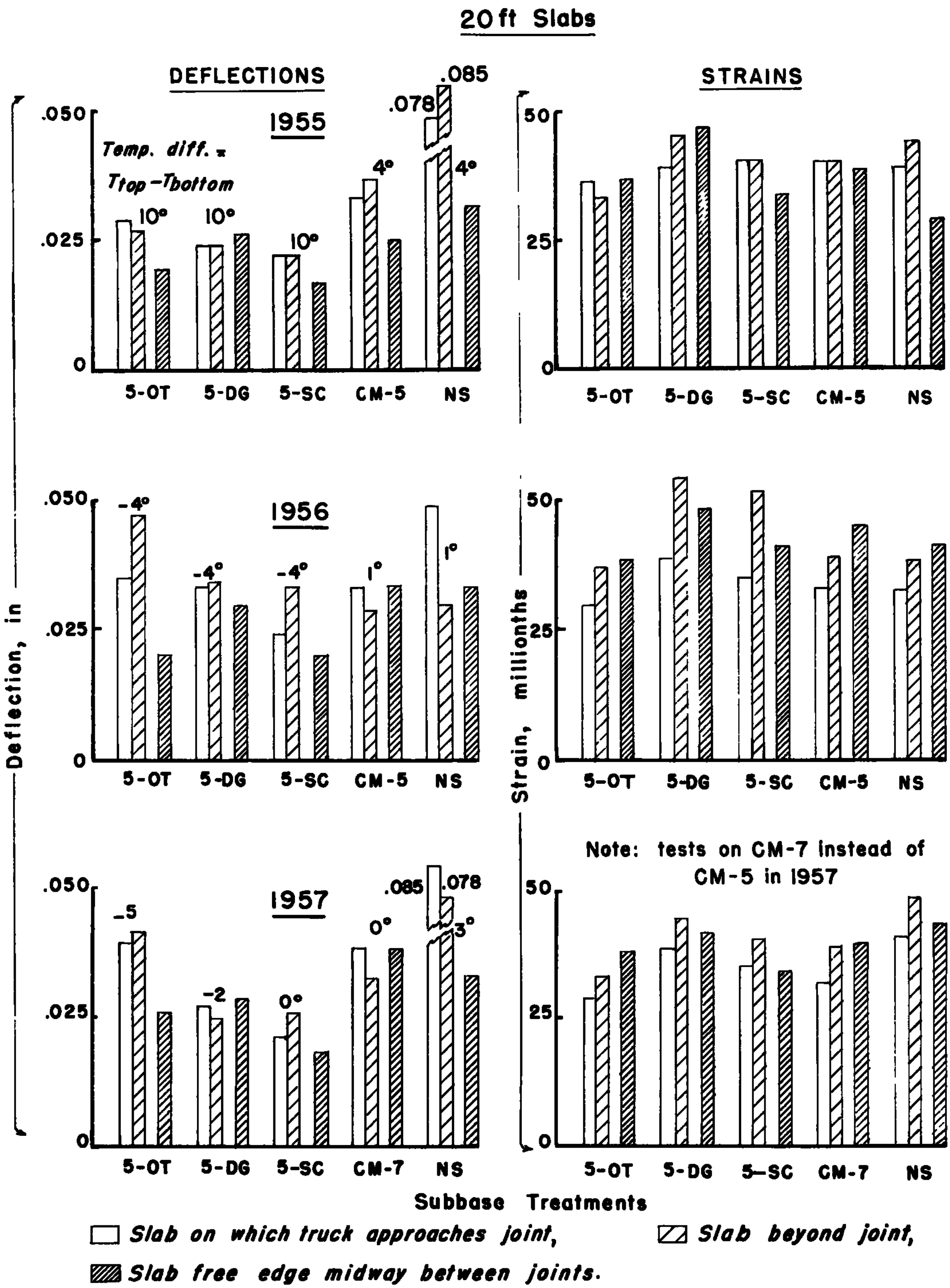


Figure 19. Maximum deflections and strains; 31,500-lb tandems moving along slab edge.

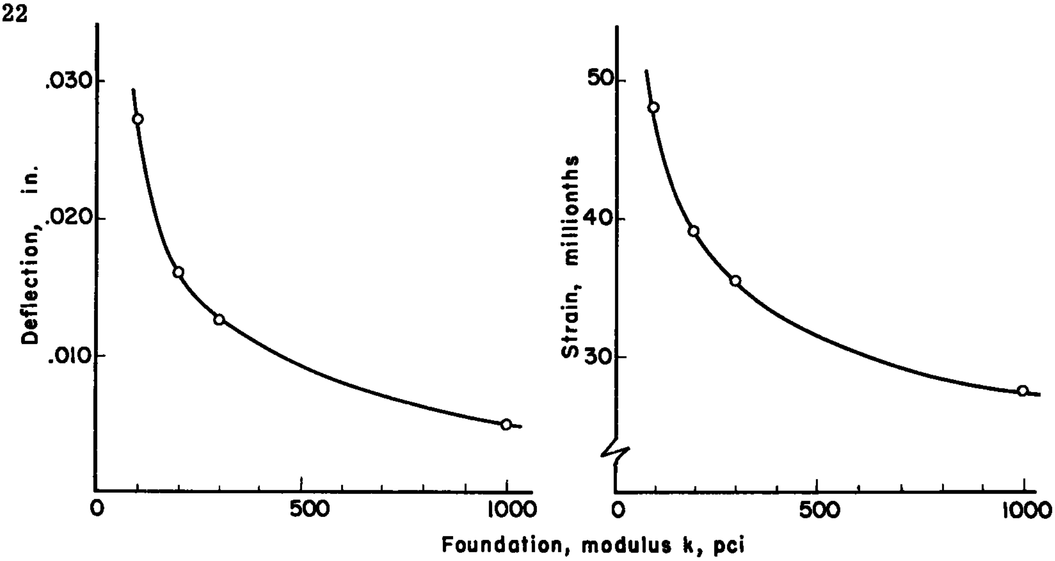


Figure 20. Theoretical deflections and strains; 31,500-lb tandems moving along free edge of 9-in. slab.

on the slab surface at mid-edge, where maximum deflections had been read in the deflection study. Compression readings in these gages were assumed to be approximately equal to the tensile strain developed directly beneath them in the bottom surface of the slab.

Linear variable differential transformers were attached to the brackets formerly used for maximum deflection measurement in order that deflections under moving loads could be correlated with measured strains.

One joint and two mid-slabs were instrumented in each specified treatment. Two deflections and two strains were read for each pass of the loading vehicle. A sample record is shown in Figure 18. The truck moved along the extreme edge at approximately 5 mph, and three passes constituted a test.

Strain and Deflection Magnitudes

A summary of maximum deflections and strains is given in the Appendix. Figure 19 is a graphic excerpt of the data from the 20-ft slab section for the last three years. It is impractical to try to compare deflection and strain magnitudes from year to year because tests were not made under the same conditions of temperature and foundation moisture content. Tests made during the same year were usually made with stable moisture content in the subbase, but often at different temperature differentials in the slabs. Thus, the moving load studies serve primarily to show the relations between deflections and strains at the time the test was made.

There is no well defined relationship between corner deflections and strains. This is explained in part by the fact that the bending radii are inversely proportional to the subgrade modulus, k. Although corner deflections on 5-OT are greater than those on 5-DG, corner strains are greater on the dense-graded material. Also, corner deflections on 5-SC are generally less than those CM-5 or CM-7. However, the corner strains on SC-5 are as great or greater than those on CM-5 or CM-7. The slabs on the NS control areas deflected much more than the others when loads were at corners, but strains were about the same as those in slabs over subbases.

At slab edges midway between joints, strains were more nearly proportional to deflections than at corners. Ranked in order of their resistance to edge deflections the subbases were 5-SC, 5-OT, 5-DG, and CM-5. Slabs on the NS control area had highest deflections. Strains were smallest over 5-SC and largest over NS, but the differences among edge strains in slabs over the other three treatments did not appear to be significant.

COMPUTATIONS

Foundation Modulus

Influence charts (4) were used to compute theoretical deflections and strains at the edge of a 9-in. concrete slab loaded by 31,500-lb tandem axles. Young's modulus for the concrete was assumed to be 4,500,000 psi, Poisson's ratio 0.15, and subgrade modulus values were arbitrarily chosen at 100, 200, 300, and 1,000 pci. These points were used to plot the curves of Figure 20, which express relations between deflection and subgrade modulus and between strain and subgrade modulus for the assumed conditions.

Estimates of foundation modulus k for each subbase were found by comparing theoretical deflections and strains with average values of those measured at the slab edges during the five years of testing. The graphs were entered with the average deflection or strain as the ordinate to find the corresponding k on the abscissa (Table 4).

Deflection data show best support from 8-OT, 5-SC and CM-9 with k=360 pci. The

TABLE 4

AVERAGE DEFLECTIONS, STRAINS, AND COMPUTED K-VALUES

Subbase		Deflection[1] (0.001 in.)				Strain[2] (millionths)			
Type	Thickness (in.)	100-ft slab	20-ft slab	Average	k[3] (pci)	100-ft slab	20-ft slab	Average	k[3] (pci)
Open-textured	3	13	15	14	250				
	5	13	11	12	320	36	36	36	275
	8	9	13	11	360				
Dense-graded	3	24	15	20	150				
	5	15	15	15	220	36	42	39	200
	8	12	13	13	280				
Soil-cement	3	13	14	14	250	35	34	34	350
	5	12	9	11	360				
Cement-modified	5	13	17	15	220				
	7	11	13	12	320	37	40	38	225
	9	12	9	11	360				
	11	14	10	12	320				
Control	0	22	20	21	130	47	43	45	120

[1] Average slab deflection under edge load; afternoon tests.
[2] Average strain at edge; moving load tests.
[3] Figure 20

lowest modulus for a subbase was k=150 pci, found in the 3-DG area. This was only a small improvement over the 130-pci value computed for the NS control.

A check on foundation modulus found by the deflection method was made using average experimental strain values and the strain-vs-k curve. The k-values by this method were in agreement for 5-DG, 5-SC, and NS, but were low for 5-OT and CM-7.

Maximum Stresses

Using a value of 4,500,000 psi for the elastic modulus of the concrete, slab stresses were computed from strain measurements. Inasmuch as all strains were measured at the edges of slabs, Poisson's ratio is not a factor in the computation and stress is the product of strain by elastic modulus. Maximum strains noted in Figure 19 for each of the subbase areas tested were 39, 54, 52, 44, and 54 millionths, respectively, for 5-OT, 5-DG, 5-SC, CM-7, and NS areas. Stresses which would produce these strains would be approximately 175, 245, 235, 200, and 245 psi. These stresses are well below 50 percent of the 600- to 630-psi modulus of rupture values noted in HRB Bulletin 116 (1).

SUMMARY AND CONCLUSIONS

Observations and tests were made for five years on a typical concrete pavement built on 12 subbase treatments including open-textured crushed stone, dense-graded crushed stone, clay soil-cement and cement-modified clay soil. Items studied were joint pumping, edge pumping, edge blowing, continuous blowing, joint faulting, transverse cracking, and slab deflections and strains under legal loads. Attempts were made to rate the subbases on their ability to prevent joint and edge pumping and blowing and on their contributions to the load carrying capacity of the pavement. Although an absolute rating of each subbase was not achieved by this experiment, the data distinguish the best and the poorest subbases, and the following conclusions are drawn:

1. Joint Pumping: The ratio of pumping contraction joints to total number of joints observed was about the same in sections with 20-ft slabs as in sections with 100-ft slabs. All thicknesses (3 in., 5 in. and 8 in.) of open-textured subbases prevented joint pumping. The dense-graded subbases resisted joint pumping, the 8-in. thickness being better than the 5-in. thickness, which in turn was better than the 3-in. thickness. The silty clay soil-cement and cement-modified subbases tended to erode under the influence of water and heavy traffic, and pumping of this slurry at joints frequently developed. The 5-in. soil-cement subbase was the best of the cement-treated subbases.

2. Edge Pumping: The severity of edge pumping was greater in sections with short slabs than in the 100-ft slab sections. Open-textured subbases prevented edge pumping almost entirely. The dense-graded treatment was effective and the thicker subbase showed a slight advantage. The cement-treated subbases showed considerable edge pumping and the 5-in. soil-cement was the best of these treatments.

3. Edge Blowing: This phenomenon was more severe along edges of short slabs than in areas with long slabs. Again the open-textured materials were most effective, but they did not prevent edge blowing. No other treatments were significantly successful in the reduction of edge blows.

4. Continuous Blowing: The severity of continuous blowing increased with time, and there were more lineal feet of blowing at the edges of short slabs than at edges of long slabs. Only the open-textured treatments were effective in restricting continuous blowing.

5. Joint Faulting: The number of faulted joints and severity of fault increased with time. Severity of fault was slightly greater between 100-ft slabs than between 20-ft slabs. Joint faulting was least on open-textured subbases, and in this group the 3-in. and 8-in. thicknesses were better than the 5-in. thickness. Next in ability to resist joint faulting were sections of 8-in. dense-graded, 5-in. and 3-in. soil-cement, and 5-in. dense-graded material. Faulting on all treatments was less than on the control area.

6. Transverse Cracks: Full depth transverse cracks were infrequent until the pavement was almost four years old. After March 1956 cracking in the right-hand lane developed rapidly. The relations among crack occurrences and subbases were not the same in sections with 20-ft slabs as in sections with 100-ft slabs: in the 20-ft slabs, fewest cracks occurred on the 5-in. soil cement and on the dense-graded subbases; in the 100-ft slabs, the dense-graded subbases were generally better than average with respect to crack resistance. There is little correlation between cracking resistance and subbase among the remaining treatments.

7. Slab Deflections: Maximum slab deflections under the loading vehicle were considerably higher during the morning tests when the slab temperature differential was low than in the afternoon when the top of the concrete was 10 to 20 degrees warmer than the bottom. This influence of slab curling and edge and corner deflections was greater on 20-ft slabs than on 100-ft slabs.

No single subbase treatment was consistently best in restricting slab deflections under all conditions of loading. In the morning tests, however, 5-in. and 8-in. dense-graded, 5-in. soil-cement, and 5-in. open-textured subbases, were best; and 8-in. open-textured, 3-in. soil-cement, and 3-in. dense-graded, were the poorest. In the afternoon tests, 5-in. soil-cement and the three open-textured subbases were among the best, and 3-in. soil-cement and 3-in. dense-graded were the poorest. Although deflections of the slab on subbases were generally less than the deflections of the slab on the control areas, under some loading conditions 3-in. soil-cement and 3-in. dense-graded subbases were less effective than the untreated subgrade soil.

8. Slab Strains: On the four areas tested, strains were smallest on slabs over 5-in. soil-cement, and were only slightly larger in 5-in. open-textured areas. Next in order were the 7 percent cement-modified and 5-in. dense-graded. Average strains in slabs on subbases were less than those in control areas.

Finally, it appears that the open-textured subbases with edge drains were the most satisfactory over-all treatments, although they were inferior to dense-graded subbases and soil-cement subbases in preventing cracking in 20-ft panels, and in restricting delections. Thickness of the open-textured subbase was not a major factor in its effectiveness and the 3-in. thickness was about as good as the 5-in. and 8-in. thicknesses. Dense-graded subbases restricted pumping and contributed to the strength of the foundation, but the 3-in. thickness was inferior to the 5-in. and 8-in. thicknesses. Subbases of soil-cement made of clay soil and subbases of cement-modified clay soil were not entirely successful because of pumping. However, 5-in. soil-cement provided a strong subbase and averaged slightly better than the other subbases from the standpoint of restricting deflections and strains and in restricting cracking in 20-ft slabs. The fact that the 5-in. clay soil-cement subbase pumped but still contributed greatly to the support of the pavement indicates that the pumping had not reached serious proportions. Nevertheless, pumping cannot be condoned and this experience suggests that clay soil-cement subbases under concrete pavements cannot be expected to control pumping until a means of preventing surface erosion is developed.

Concrete aprons and gutters adjoining the paving slabs prevented edge blowing and pumping and were very effective in the restriction of joint pumping. Subbases in areas which were built with gutters and aprongs could not be classified as to performance because of the universal effectiveness of these adjuncts over all treatments.

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Appendix

TABLE 5

TRAFFIC SUMMARIES

(a) 24-HOUR WEEKDAY TRAFFIC COUNT

Month		Light Vehicles: Pass. Cars	Light Vehicles: Panels and Pickups	Heavy Vehicles: Single Unit Trucks	Heavy Vehicles: Combination Trucks	Heavy Vehicles: Total	Total Vehicles
February	1955	2555	197	265	2407	2672	5424
May	1955	3783	345	322	2251	2573	6701
August	1955	5595	223	334	2076	2410	8228
September	1955	5341	302	280	2042	2322	7965
October	1955	2288	190	315	1802	2117	4595
November	1955	2174	172	192	1836	2028	4374
December	1955	2584	154	222	2002	2224	4962
Avg.	1955	3474	226	276	2060	2336	6036
February	1956	2062	167	197	1841	2038	4267
June	1956	2684	209	214	1704	1918	4811
August	1956	2886	205	192	1404	1596	4687
December	1946	2455	177	219	1619	1838	4470
Avg.	1956	2522	189	205	1642	1848	4559
February	1957	2223	163	189	1649	1838	4224
September	1957	2665	214	331	1428	1759	4638
Avg.	1957	2444	188	260	1539	1799	4431

(b) TOTAL TRUCK AXLE FREQUENCY BY VARIOUS WEIGHT GROUPS FOR A 24-HOUR WEEKDAY

Weight Groups 1,000 lb	Total Number of Axles													
	1954		1955						1956				1957	
	Nov.	Dec.	Feb.	May	Aug.	Sept.	Oct.	Nov.	Feb	June	Aug.	Dec.	Feb.	Sept.
No Pay Load	2391	3570	3954	3750	3474	3536	3117	3036	2810	2557	2069	2821	2703	2136
Under 14	3015	3314	3730	3627	3579	3323	3056	2997	3278	3102	2588	2623	2774	2996
14 - 15	444	550	547	520	454	455	446	515	380	394	308	349	368	266
15 - 16	365	433	427	570	473	434	361	365	353	285	230	244	288	242
16 - 17	356	288	259	317	270	278	262	228	274	207	208	202	231	231
17 - 18	267	188	250	238	200	240	181	170	179	229	157	200	214	183
18 - 19	168	145	238	204	169	158	120	104	122	154	124	154	104	142
19 - 20	102	81	127	81	65	104	56	69	45	68	85	104	95	61
20 - 21	16	26	33	15	18	34	9	8	12	23	10	20	24	20
21 - 22	-	9	20	7	3	10	3	5	-	3	9	2	3	7
22 - 23	-	2	6	-	-	-	-	-	3	5	2	-	-	2
23 - 24	-	-	2	-	-	2	-	3	-	-	-	-	-	-
24 and over	-	-	-	-	-	-	-	-	-	-	-	-	-	2
Total Axles	7114	8606	9593	9329	8705	8574	7611	7500	7456	7027	5788	6719	6804	6288

(c) SINGLE TRUCK AXLE FREQUENCY BY WEIGHT GROUPS FOR A 24-HOUR WEEKDAY

Axle Weight Groups 1,000 lb	Number of Single Axles													
	1954		1955						1956				1957	
	Nov.	Dec.	Feb.	May	Aug.	Sept	Oct.	Nov.	Feb.	June	Aug.	Dec.	Feb.	Sept.
No Pay Load	1699	2718	3176	2874	2690	2754	2403	2310	2144	1917	1457	2069	2015	1510
Under 14	1708	2320	2592	2207	2316	1960	1871	1799	1996	1893	1459	1720	1619	1642
14 - 15	235	390	412	352	309	296	318	412	281	248	186	231	234	154
15 - 16	250	341	329	472	387	335	307	279	283	227	183	172	227	188
16 - 17	309	265	233	271	236	249	244	205	254	189	172	173	186	194
17 - 18	235	173	236	236	188	236	165	159	169	217	147	172	206	157
18 - 19	168	139	226	204	159	144	109	93	102	139	116	129	96	127
19 - 20	98	73	117	79	53	92	53	69	42	65	77	93	81	58
20 - 21	16	26	31	13	15	30	6	8	12	20	10	20	16	17
21 - 22	-	1	17	7	-	10	3	5	-	3	7	2	-	7
22 - 23	-	-	6	-	-	-	-	-	3	5	2	-	-	2
23 - 24	-	-	2	-	-	2	-	3	-	-	-	-	-	-
24 and over	-	-	-	-	-	-	-	-	-	-	-	-	-	2
Total	4718	6446	7377	6715	6353	6108	5479	5342	5286	4923	3816	4781	4680	4058

(d) TANDEM TRUCK AXLE FREQUENCY BY WEIGHT GROUPS FOR A 24-HOUR WEEKDAY

Axle Weight Groups 1,000 lb	Number of Tandem Axles, Listed Individually													
	1954		1955						1956				1957	
	Nov.	Dec.	Feb.	May	Aug.	Sept.	Oct.	Nov.	Feb.	June	Aug.	Dec.	Feb.	Sept.
No Pay Load	692	852	778	876	784	782	714	726	666	640	612	752	688	626
Under 14	1307	994	1138	1420	1263	1363	1185	1198	1282	1209	1129	905	1155	1354
14 - 15	209	160	135	168	145	159	126	103	99	146	122	118	134	112
15 - 16	115	92	98	98	86	99	54	86	70	58	47	71	61	54
16 - 17	47	23	26	46	34	29	18	23	20	18	36	28	45	37
17 - 18	22	15	14	2	12	4	16	11	10	12	10	28	8	26
18 - 19	-	6	12	-	10	14	11	11	20	15	8	25	8	15
19 - 20	4	8	10	2	12	12	3	-	3	3	8	11	14	3
20 - 21	-	-	2	2	3	-	3	-	-	3	-	-	8	3
21 - 22	-	8	3	-	3	-	-	-	-	-	-	-	3	-
22 - 23	-	2	-	-	-	-	-	-	-	-	-	-	-	-
23 - 24	-	-	-	-	-	-	-	-	-	-	-	-	-	-
24 and over	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total	2396	2160	2216	2614	2352	2466	2132	2158	2170	2104	1972	1938	2124	2230

TABLE 6

NUMBER OF PUMPING JOINTS PER 100 FEET OF UNCURBED LENGTH

Sub-base	1953		1954	1955		1956			1957
	Mar.	May.	Mar.	Apr.	Oct.	Jan.	Mar.	Dec.	May
(a) 20-Ft Slabs									
3-OT	-	-	-	-	-	-	-	-	-
5-OT	-	-	-	-	-	-	-	-	-
8-OT	-	-	-	-	-	-	-	-	-
3-DG	-	-	-	0.3	-	-	-	0.5	0.1
5-DG		-	-	0.1	-	-	-	-	-
8-DG	-	0.1	0.2	-	-	-	-	-	-
3-SC	-	-	0.9	1.2	0.2	-	1.9	3.9	0.3
5-SC	-	-	0.7	0.9	0.3	-	2.2	3.3	0.2
CM-5	-	0.6	4.2	2.8	0.9	-	3.4	4.7	0.7
CM-7	-	0.6	2.5	2.9	1.2	0.1	4.0	4.1	0.9
CM-9	0.1	0.3	1.7	1.2	1.3	-	1.5	4.7	0.5
CM-11	-	0.8	1.6	0.4	0.2	-	2.0	3.4	-
NS	0.7	1.3	7.0	4.6	3.0	0.7	4.7	5.0	3.0
(b) 100-Ft Slabs									
3-OT	-	-	-	-	-	-	-	-	-
5-OT	-	-	-	-	-	-	-	-	-
8-OT	-	-	-	-	-	-	-	-	-
3-DG	-	-	-	0.8	-	-	0.4	-	-
5-DG	-	-	-	0.2	-	-	-	0.2	-
8-DG	-	-	-	-	-	-	-	-	-
3-SC	-	0.6	1.1	0.8	-	-	0.8	0.6	-
5-SC	-	0.2	0.4	0.8	-	-	0.6	0.4	0.4
CM-5	-	0.5	0.5	0.6	-	-	0.5	0.3	-
CM-7	-	-	0.7	0.5	-	-	0.5	0.5	-
CM-9	-	-	0.2	0.3	0.1	-	0.2	0.3	-
CM-11	-	0.2	0.7	0.7	-	-	0.5	0.1	-
NS	0.1	0.1	0.7	0.8	0.1	0.1	0.7	0.1	0.7

TABLE 7

OCCURRENCES OF EDGE PUMPING PER 100 FEET OF UNCURBED LENGTH

Sub-base	1953		1954	1955		1956			1957
	Mar.	May	Mar.	Apr.	Oct.	Jan.	Mar.	Dec.	May
(a) 20-Ft Slabs									
3-OT	-	-	-	-	-	-	-	-	-
5-OT	-	-	-	-	-	-	-	-	-
8-OT	-	-	-	-	-	-	-	-	-
3-DG	-	0.9	0.4	0.1	0.1	-	0.1	-	-
5-DG	-	-	-	-	-	-	-	-	-
8-DG	-	0.4	0.1	-	-	-	-	-	-
3-SC	-	1.4	2.7	3.5	3.8	1.8	5.7	5.2	3.0
5-SC	-	1.0	2.5	1.3	3.1	1.3	6.6	5.6	4.2
CM-5	0.3	3.4	3.5	4.5	5.4	1.2	8.8	6.5	2.9
CM-7	9.5	5.1	2.6	5.3	4.4	1.2	10.0	6.0	5.8
CM-9	0.4	3.5	1.2	4.2	4.6	0.7	4.0	3.1	4.2
CM-11	0.2	1.6	1.0	2.6	4.2	0.8	9.0	6.4	6.6
NS	-	1.7	3.4	7.7	8.7	7.4	10.0	7.7	7.4
(b) 100-Ft Slabs									
3-OT	-	-	-	-	1.4	-	-	-	-
5-OT	-	-	-	-	1.0	-	-	-	-
8-OT	-	-	-	-	0.4	-	-	-	-
3-DG	-	-	-	0.4	4.4	-	-	-	-
5-DG	-	-	-	0.2	3.4	-	-	-	-
8-DG	-	-	-	-	0.4	-	-	-	-
3-SC	-	1.1	0.8	0.8	0.3	0.3	0.3	-	1.4
5-SC	0.1	0.4	0.4	0.8	0.6	0.2	-	-	-
CM-5	-	1.0	0.9	0.3	1.1	0.3	0.3	-	0.2
CM-7	-	1.7	2.4	0.6	1.1	0.4	0.2	-	0.2
CM-9	-	0.5	1.4	0.3	1.4	0.6	0.2	0.1	0.1
CM-11	-	0.6	0.7	0.2	0.5	0.1	0.2	-	-
NS	0.1	0.9	1.4	0.8	3.4	0.2	0.5	0.2	1.0

TABLE 8

NUMBER OF BLOWHOLES AT EDGE PER 100 FEET OF UNCURBED LENGTH

Sub-base	1953		1954	1955		1956			1957
	Mar.	May	Mar.	Apr.	Oct.	Jan.	Mar.	Dec.	May
(a) 20-Ft Slabs									
3-OT	0.1	0.3	4.6	2.9	10.2	3.5	12.2	0.7	0.9
5-OT	0.2	1.7	2.4	1.0	15.5	7.2	-	0.7	0.2
8-OT	0.3	1.1	0.1	-	1.8	-	-	0.4	-
3-DG	-	6.8	11.9	40.3	44.7	13.5	30.4	10.0	2.6
5-DG	0.4	11.5	18.9	27.8	57.1	10.5	47.8	17.7	3.0
8-DG	1.1	13.3	17.8	23.6	26.8	10.3	49.8	10.9	1.7
3-SC	1.3	16.3	27.8	23.0	20.7	9.7	19.0	7.1	0.4
5-SC	0.8	20.6	12.1	20.6	24.6	7.0	31.4	11.6	5.5
CM-5	2.0	14.2	5.6	18.8	23.6	12.5	5.6	11.8	-
CM-7	9.1	10.0	12.8	20.5	7.5	9.6	9.1	5.4	-
CM-9	10.9	22.6	25.4	21.7	17.0	10.2	17.1	8.5	0.1
CM-11	2.8	13.2	13.0	27.0	25.4	5.6	10.0	5.2	6.2
NS	4.4	19.5	3.4	14.1	-	-	-	2.0	-
(b) 100-Ft Slabs									
3-OT	-	-	0.2	-	-	-	0.4	-	-
5-OT	-	-	2.2	0.2	-	-	1.4	-	-
8-OT	-	-	1.6	0.6	-	0.2	1.6	-	-
3-DG	-	0.8	10.2	8.4	6.8	4.0	-	3.2	4.4
5-DG	-	-	12.6	4.6	0.4	11.0	2.0	1.4	16.4
8-DG	-	3.0	18.4	6.2	0.8	7.6	1.8	5.4	10.0
3-SC	0.6	2.2	12.3	5.2	-	-	-	1.1	0.3
5-SC	0.9	2.9	3.4	2.4	-	0.4	-	-	5.0
CM-5	0.3	2.9	4.6	1.2	1.4	-	0.3	0.8	0.9
CM-7	1.5	9.8	2.1	3.5	-	0.6	0.1	6.4	-
CM-9	0.6	14.2	0.9	5.7	0.3	-	0.7	2.8	2.2
CM-11	4.1	9.6	8.0	3.9	0 2	-	0.2	1.6	0.4
NS	0.1	1.7	1.6	1.8	-	0.1	1.7	0.5	0.3

TABLE 9

LINEAL FEET OF BLOWING PER 100 FEET OF UNCURBED LENGTH

Sub-base	1953		1954	1955		1956			1957
	Mar.	May	Mar.	Apr.	Oct.	Jan.	Mar.	Dec.	May
(a) 20-Ft Slabs									
3-OT	-	-	-	-	-	0.5	-	-	2.8
5-OT	-	-	-	-	-	-	-	-	-
8-OT	-	-	-	-	-	-	-	-	-
3-DG	-	1.0	-	-	1.4	-	17.1	1.0	2.7
5-DG	-	1.7	-	-	-	-	12.8	15.1	2.2
8-DG	-	4.4	3.2	3.0	9.3	-	16.0	9.0	1.8
3-SC	-	1.8	-	5.9	18.8	10.1	7.8	3.3	23.2
5-SC	-	8.3	11.5	24.5	13.7	1.0	16.4	3.2	11.6
CM-5	-	6.1	2.1	4.5	11.1	14.5	9.2	21.6	22.9
CM-7	1.1	17.0	19.9	9.4	39.7	22.5	2.8	32.4	39.1
CM-9	0.3	24.9	8.4	4.2	4.4	7.5	30.8	31.5	49.1
CM-11	-	-	5.0	13.2	-	4.6	9.0	11.8	-
NS	-	-	8.7	-	-	-	-	-	16.1
(b) 100-Ft Slabs									
3-OT	-	-	-	-	12.0	-	-	-	-
5-OT	-	-	-	-	4.0	-	1.0	-	-
8-OT	-	-	-	-	5.0	1.0	2.0	-	2.0
3-DG	-	1.2	-	33.0	-	6.0	100.0	4.0	10.0
5-DG	-	-	-	18.0	0.6	2.0	69.0	-	2.0
8-DG	-	-	-	30.0	-	4.0	80.0	-	1.0
3-SC	-	11.0	2.2	0.8	23.4	5.5	64.8	2.8	11.8
5-SC	-	2.5	0.6	-	4.0	2.0	15.0	-	9.0
CM-5	-	5.1	1.1	2.3	6.0	2.0	27.5	2.7	3.9
CM-7	-	1.3	2.5	1.5	8.0	5.5	43.5	-	6.5
CM-9	-	1.2	0.8	-	3.0	1.2	54.0	2.4	9.8
CM-11	-	6.7	1.0	3.0	19.0	3.0	27.5	2.0	7.5
NS	-	-	1.3	1.7	-	0.5	4.5	-	2.0

TABLE 10

JOINT FAULTING

Sub-base	Number of Faults in Each Class[a]																	
	March 1954			April 1955			March 1956			May 1956			Dec 1956			May 1957		
	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
(a) 20-Ft Slabs																		
3-OT	1	-	-	6	-	-	25	6	1	23	10	-	31	12	-	27	10	-
5-OT	1	1	-	6	1	-	18	7	-	20	6	-	28	8	-	23	7	-
8-OT	4	1	-	9	1	-	18	6	-	25	5	-	26	9	-	24	9	-
3-DG	8	3	-	7	7	-	13	24	2	11	28	4	12	31	3	16	31	2
5-DG	2	2	-	7	3	-	13	21	1	16	25	2	23	21	3	19	25	2
8-DG	3	-	-	7	1	-	15	21	1	20	24	2	15	27	3	20	22	3
3-SC	2	-	-	5	2	-	17	16	-	15	24	1	16	22	2	17	19	3
5-SC	3	1	-	6	2	-	15	15	-	26	16	-	19	20	-	18	24	1
CM-5	7	4	-	8	8	-	19	16	4	17	24	2	15	26	3	8	28	6
CM-7	6	1	-	13	4	-	17	16	4	19	20	1	13	22	3	13	25	4
CM-9	5	3	-	10	8	-	17	20	6	18	25	3	16	27	1	10	28	9
CM-11	3	2	-	7	3	-	6	9	1	6	13	-	8	13	-	6	13	1
NS	4	2	-	7	7	-	10	8	1	9	11	1	6	15	1	6	14	3
(b) 100-Ft Slabs																		
3-OT	1	-	-	-	-	-	2	1	-	4	15	-	3	2	-	4	1	-
5-OT	1	-	-	3	-	-	7	3	-	6	5	-	7	3	1	7	4	-
8-OT	1	-	-	1	3	-	5	1	1	6	-	1	5	2	1	5	2	1
3-DG	-	-	-	4	-	-	4	5	1	2	7	1	1	8	1	1	8	1
5-DG	-	-	-	3	-	-	4	4	-	2	6	-	1	7	-	1	7	-
8-DG	2	-	-	2	1	-	3	3	2	4	4	-	4	5	-	3	5	1
3-SC	1	-	-	1	3	-	2	5	-	3	5	-	3	5	-	2	5	1
5-SC	-	-	-	2	-	-	6	1	-	6	2	-	2	6	-	3	6	-
CM-5	-	-	-	-	1	-	2	6	-	5	4	1	4	5	1	2	8	-
CM-7	1	-	-	-	2	-	3	5	2	2	5	2	3	5	2	1	7	2
CM-9	1	-	-	1	2	-	1	3	1	3	3	2	3	4	1	3	4	1
CM-11	1	-	-	2	2	-	2	5	1	-	8	1	2	7	1	2	7	1
NS	2	1	1[b]	1	3	1	5	2	1	4	5	1	4	3	-	2	3	2

[a] Class 1 = 1/16 in ; Class 2 = 1/8 to 3/16 in.; Class 3 = 1/4 in and greater.
[b] Broken slab

TABLE 11

TRANSVERSE CRACKS

Sub-base	Total Count of Transverse Cracks						
	Mar 1954	Apr 1955	Oct 1955	Jan 1956	Mar 1956	Dec 1956	May 1957
(a) 20-Ft Slabs[a]							
3-OT	-	-	-	-	-	7	20
5-OT	-	-	-	-	-	10	22
8-OT	-	-	1	1	1	16	33
3-DG	-	-	2	2	2	4	8
5-DG	-	-	1	1	1	6	12
8-DG	-	-	-	-	-	-	6
3-SC	-	-	-	1	2	16	19
5-SC	-	-	-	-	-	3	5
CM-5	-	1	2	2	2	19	22
CM-7	-	1	2	3	3	18	24
CM-9	-	-	4	4	8	22	36
CM-11[b]	-	-	-	-	-	8	12
NS[b]	-	-	3	3	4	24	27
(b) 100-Ft Slabs[c]							
3-OT	1	1	4	9	12	34	44
5-OT	1	2	11	11	13	28	50
8-OT	3	5	5	10	10	35	53
3-DG	5	8	9	10	10	23	41
5-DG	-	-	2	2	2	31	35
8-DG	1	4	7	15	15	30	43
3-SC	-	-	-	8	8	27	64
5-SC	-	1	1	4	4	16	60
CM-5	1	1	2	4	4	39	63
CM-7	-	1	2	7	7	44	68
CM-9	1	3	8	13	14	49	66
CM-11	-	1	7	10	12	31	55
NS	7[d]	8	10	11	12	25	32

[a] 50 slabs each treatment
[b] 25 slabs
[c] 10 slabs each treatment.
[d] Undersealed.

TABLE 12-A

MAXIMUM DEFLECTIONS,

31,500-LB TANDEM AXLE ALONG OUTER EDGE—AFTERNOON TESTS

Sub-base	Average Deflection (0 001 in)														
	Doweled Corner[a]										Free Edge				
	1953		1954		1955		1956		1957		1953	1954	1955	1956	1957
	An	Lv	Ap	Lv	Ap.	Lv	Ap	Lv	Ap.	Lv					
(a) 20-Ft Slabs															
ΔT[b]	15 deg		10 deg		22 deg		13 deg		7 deg		15 deg	10 deg	22 deg	13 deg	7 deg
3-OT	9	10	28	25	21	17	13	15	35	26	11	17	15	16	15
5-OT	9	8	18	18	9	10	15	14	22	20	9	10	9	14	11
8-OT	12	13	23	22	14	14	13	12	18	16	14	16	10	13	12
3-DG	14	14	21	17	13	13	10	7	18	17	15	17	12	15	15
5-SG	12	10	16	14	11	9	14	9	18	17	13	13	13	16	19
8-DG	13	11	18	22	11	14	14	15	14	10	10	13	13	17	14
3-SC	10	11	22	25	15	13	19	16	26	25	11	15	12	15	15
5-SC	8	8	18	13	6	6	8	8	17	12	8	10	9	10	10
CM-5	6	10	25	21	15	15	11	14	30	30	14	19	16	17	17
CM-7	8	7	19	15	10	9	9	8	19	19	13	15	12	11	14
CM-9	9	7	14	19	9	13	8	10	15	24	11	10	9	8	9
CM-11	8	10	15	23	12	11	15	9	21	25	8	11	9	11	12
NS	24	21	32	39	29	31	18	19	58	47	22	20	21	19	20
(b) 100-Ft Slabs															
ΔT[b]	12 deg		12 deg		21 deg		14 deg		13 deg		12 deg	12 deg	21 deg	14 deg	13 deg
3-OT	20	19	21	17	6	9	7	9	9	7	20	13	12	11	8
5-OT	15	14	13	14	13	14	14	13	14	11	16	14	15	9	10
8-OT	20	13	11	13	15	14	9	11	10	11	9	10	10	9	7
3-DG	38	37	32	27	24	24	33	30	20	21	20	21	27	28	23
5-DG	25	23	15	14	16	16	22	19	13	13	9	15	16	17	15
8-DG	26	25	18	20	14	14	18	15	10	14	12	14	14	8	13
3-SC	21	24	34	43	23	27	11	11	10	5	13	15	12	14	12
5-SC	12	-	18	17	10	10	13	8	11	9	11	14	12	15	9
CM-5	16	15	14	10	13	11	21	17	18	13	12	11	13	15	13
CM-7	15	15	12	17	9	6	13	14	18	14	12	11	10	13	11
CM-9	14	12	15	14	16	14	19	22	16	17	12	11	13	15	10
CM-11	15	12	20	18	12	16	25	32	16	18	12	14	13	17	16
NS	102	95	73	83	27	26	62	62	41	42	20	21	23	26	21

[a] Ap. = Slab on which truck approaches joint, Lv = Slab on which truck leaves joint

[b] Temperature differential, F.

TABLE 12 B

MAXIMUM DEFLECTIONS,

31,500-LB TANDEM AXLE ALONG OUTER EDGE—MORNING TESTS

Sub-base	Average Deflection (0 001 in)														
	Doweled Corner										Free Edge				
	1953		1954		1955		1956		1957		1953	1954	1955	1956	1957
	Ap	Lv	Ap	Lv.	Ap.	Lv	Ap	Lv	Ap	Lv					
(a) 20-Ft Slabs															
ΔT	-2 deg		-4 deg		4 deg		6 deg		4 deg		2 deg	0 deg	4 deg	6 deg	4 deg
3-OT	-	-	34	35	49	48	30	47	37	32	18	20	29	19	14
5-OT	-	-	23	33	45	61	31	37	26	24	14	16	27	13	13
8-OT	-	-	44	49	54	53	37	44	27	33	23	24	39	20	14
3-DG	-	-	28	44	27	27	34	30	26	19	18	20	21	16	17
5-DG	-	-	29	30	26	34	26	28	20	22	15	17	26	20	18
8-DG	-	-	25	31	20	22	25	27	13	12	14	19	22	21	14
3-SC	-	-	37	44	49	40	40	40	30	33	16	21	25	21	20
5-SC	-	-	39	41	42	47	18	25	21	26	14	18	33	15	11
CM-5	43	43	41	34	37	37	33	29	33	46	25	27	30	27	18
CM-7	34	34	27	30	47	46	26	24	21	20	21	24	22	16	15
CM-9	22	21	21	39	31	42	16	30	18	33	11	15	19	12	10
CM-11	-	-	26	36	46	45	30	38	23	28	21	17	32	14	19
NS	47	46	47	60	84	85	45	49	62	52	20	20	30	23	20
(b) 100-Ft Slabs															
ΔT	-2 deg		0 deg		-1 deg		3 deg		1 deg		1 deg	-1 deg	-1 deg	3 deg	1 deg
3-OT	-	-	41	35	61	56	45	43	24	19	26	29	39	26	12
5-OT	-	-	30	38	43	43	32	39	19	19	23	32	44	24	17
8-OT	-	-	30	42	70	59	34	39	17	18	13	23	36	24	15
3-DG	-	-	59	61	51	53	53	52	39	29	19	24	34	35	24
5-DG	-	-	26	31	43	41	47	42	17	18	17	20	29	21	21
8-DG	-	-	37	36	37	47	35	35	30	35	18	23	29	26	30
3-SC	45	49	50	75	74	102	22	16	20	21	15	18	30	21	19
5-SC	26	24	25	34	46	41	28	27	20	19	12	16	30	19	16
CM-5	28	27	26	29	41	39	41	50	29	26	14	13	24	18	20
CM-7	26	29	25	39	40	35	31	36	19	24	17	17	27	21	18
CM-9	34	31	33	43	68	64	41	43	25	24	15	15	36	19	17
CM-11	39	36	30	28	53	69	47	53	38	41	17	15	29	22	22
NS	-	-	92	113	75	87	93	94	68	75	18	25	39	33	28

TABLE 13

MAXIMUM DEFLECTIONS AND STRAINS UNDER MOVING LOADS

Sub- base	20-Ft Slabs							100-Ft Slabs						
	Temp. Diff.	Defl. (0.001 in.)			Strain (millionths)			Temp. Diff.	Defl. (0.001 in.)			Strain (millionths)		
		Corner[a]		Edge	Corner[a]		Edge		Corner[a]		Edge	Corner[a]		Edge
	F	Ap.	Lv.		Ap.	Lv.		F	Ap.	Lv.		Ap.	Lv.	
							1953							
5-OT	4	15	16	12	27	26	29	11	13	11	16	28	37	38
5-DG	4	18	16	25	33	30	41	6	14	16	12	32	32	31
5-SC	-2	54	34	23	41	39	31	8	22	24	21	33	34	38
CM-7	0	26	26	21	28	28	36	15	36	26	23	33	31	39
NS	0	31	30	22	41	40	36	9	61	60	15	33	47	39
							1954							
5-OT	5	29	25	-	27	23	-	6	23	26	15	27	26	31
5-DG	8	11	18	20	34	26	31	6	21	31	12	29	27	25
5-SC	4	24	28	-	25	29	-	2	23	28	23	30	29	28
CM-7	4	22	24	-	29	28	-	2	20	22	14	32	30	-
NS	6	36	48	-	31	33	-	8	57	75	27	38	40	39
							1955							
5-OT	10	28	27	18	36	33	36	15	14	15	15	33	30	37
5-DG	10	24	24	27	38	45	47	15	26	25	18	43	37	38
5-SC	10	22	22	17	40	40	33	12	24	25	20	37	37	38
CM-5	4	33	37	25	40	40	38	12	23	22	19	32	33	31
NS	4	78	85	32	38	44	48	15	32	32	28	47	45	47
							1956							
5-OT	-4	35	47	20	30	37	38	3	23	24	15	24	31	-
5-DG	-4	33	34	30	39	54	48	3	47	44	28	27	-	-
5-SC	-4	24	33	20	35	52	41	3	24	22	18	28	31	32
CM-5	1	30	33	28	33	39	45	3	25	42	17	37	38	34
NS	1	51	48	30	33	38	42	3	87	90	30	31	-	53
							1957							
5-OT[b]	-5	40	42	26	29	33	38	0	33	62	35	32	38	39
5-DG[b]	-2	27	25	28	39	45	42	0	80	55	58	38	48	48
5-SC[b]	0	21	26	18	35	41	34	3	20	19	25	35	33	42
CM-7[b]	0	39	33	39	32	39	40	5	34	29	29	36	44	44
NS[b]	3	85	78	33	41	48	44	5	102	68	103	40	54	50

[a] Ap. = Slab on which truck approaches joint; Lv. = Slab on which truck leaves joint.
[b] In 1957 the subbases tested under 100-ft slabs were 3-OT, 3-DG, 5-SC, and CM-9.

Performance of Subbases for Concrete Pavements Under Repetitive Loading

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Laboratory data are reported on the performance of subbases for concrete pavements under the action of 500,000 repetitions of load. The type and gradation of subbase materials, and the placement conditions relative to density and moisture content, were evaluated.

Nine granular subbase materials ranging in gradation from an open-graded, free-draining material to a dense-graded, low-permeability material were tested. The materials were placed at densities ranging from approximately 80 percent to 110 percent of standard AASHO density, and at moisture contents ranging from 75 percent to 120 percent of AASHO optimum. The subbase thickness was 6 in. except for one material where the thickness ranged from 1 in. to 12 in. Four granular soil-cement subbases were tested.

The subbases were placed on a $2\frac{1}{2}$ ft thick clay subgrade confined in a concrete box 4 ft by 6 ft by 3 ft deep. A 2-in. prestressed concrete slab, jointed at mid-length, was cast on the subbase and loads were applied over the joint at the rate of about 20 applications per minute. The loads were of sufficient magnitude to cause a pressure on the subgrade of about 7 psi. During the repetitive load operations, the moisture content of the subbase was increased from the placement moisture to a condition of saturation. The amount of densification that occurred, both in the subbase layer and in the composite subbase-subgrade foundation, was measured.

The data show that, for the materials investigated, an increase in the placement density from 90 to 108 percent of standard AASHO density reduced subbase densification by 80 percent, and an increase in density from 90 to 100 percent reduced densification by 45 percent. These results suggest that granular subbases should be compacted to at least 100 percent of AASHO standard density to preclude harmful densification under traffic.

Under the extremely severe conditions of these laboratory tests, the performance of certain dense-graded subbases was adversely affected by pumping, and the performance of open-graded, high-permeability material was adversely affected by intrusion of subgrade soil into the subbase. A 1-in. filter course of sand or of dense-graded crushed stone prevented the intrusion.

The densification of granular soil-cement subbases was practically zero, and the performance of the subbases was excellent.

●SUBBASES of granular materials are used under concrete pavements to fulfill three principal purposes: to provide protection against the attendant problems of frost action, to restrict volume change of certain subgrade soils, and to prevent pumping. In addition, when properly placed and compacted, subbases distribute the load to the subgrade and provide uniform and stable support for the pavement.

The use of subbases for prevention of pumping gained importance with the advent of more frequent and heavier wheel loads during the early 1940's. Pumping and the attendant loss of subgrade support, which in some cases led to structural failure of the pavement, became a widespread and serious problem. As a result, a committee to study pumping was appointed by the Highway Research Board in 1942, and in 1948 the final report of this committee was published (1). The report stated that subbases of granular materials placed over fine-grained subgrade soils would prevent pumping.

However, it was considered that more research was necessary to show the influence of subbase thickness and of subbase gradation and type in the prevention of pumping. Furthermore, field studies and observations have indicated two other principal areas of subbase design, beyond the function of inhibiting pumping, where additional research was required.

One of the areas is related to the strength of the pavement structure and suggests an investigation of the influence of various types and thicknesses of subbase materials on the load-carrying capacity of the pavement. The Portland Cement Association has in progress an investigation of this phase of subbase design, and data obtained from the first series of laboratory tests have been reported (2).

The second area concerns the densification of subbases under the action of repetitive loads, and it is with this phase of the problem that this report is concerned. Highway traffic operations tend to densify or consolidate the subbase material, resulting in a settlement of the pavement. If this settlement were uniform, little damage would occur, but usually more settlement occurs at transverse joints than elsewhere. Eventually the settlement may cause faulted joints and in severe cases a structural failure of the pavement slab. It is known that increasing the placement density of a material decreases the magnitude of further densification that may occur under repeated loadings, but little quantitative data are available on this subject.

The main purpose of this laboratory investigation was to determine the effects of subbase type, gradation, placement density, and placement moisture content on the densification of the subbase under repetitive loading. The ultimate aim of the project was to obtain experimental data which would aid in writing a specification controlling initial compaction of subbases to preclude harmful densification of the subbase in service. Supplemental information was obtained on the influence of thickness and physical characteristics of subbases in preventing pumping.

PLAN AND SCOPE OF PROGRAM

Nine granular subbases and 4 soil-cement subbases were tested. They were compacted on a clay subgrade soil which had been placed at AASHO standard density and optimum moisture in a 4- by 6- by 3-ft rigid container. For each test, a concrete slab was built on the subbase, and through this slab 500,000 loads were applied to the subbase-subgrade foundation. Water was added at various stages of the loading plan until the subbase was saturated. Measurements were made to determine the progressive densification of the subbase and of the subgrade. Measurements were also made of the pressure on the subgrade. Observations were made of the pumping phenomenon if it occured at various stages of the repetitive load program.

The principal variables of the program of tests on the granular subbases included (a) the initial in-place density of the subbase, ranging from about 80 to 110 percent of AASHO[1] standard; (b) initial in-place moisture content, ranging from 75 to 120 percent of AASHO optimum; (c) the type of subbase, principally sand and gravel and crushed limestone; and (d) the gradation of the subbase. Secondary variables included the thickness of the granular subbase layer and various combinations or blends of the subbase materials. The soil-cement subbases were tested only at AASHO standard density and optimum moisture content.

Supplementary tests were performed to aid in evaluating the performance of the granular subbases. These tests included minimum and maximum density, permeability, and triaxial compression.

MATERIALS

Subbases

The subbases included 5 basic materials selected to represent a range in gradation frequently used in current subbase construction plus 4 supplementary materials

[1] AASHO designation: T-99, The Compaction and Density of Soils.

which were made by changing the gradation of one of the basic materials by the addition or omission of fines. Table 1 gives a brief description of each material, together with the number which is used for identification throughout the text. Materials 1 through 5 are the basic materials, and 6 through 9 comprise the supplementary materials. Subbases of soil-cement were made of materials 1 through 4. A subbase thickness of 6 in. was used in testing all materials, and in addition, tests on material 1 included thicknesses of 1, 2, 4 and 12 in. The gradations of materials are shown in Figures 1a and 1b. With the exception of material 2 which had a PI of 2, the soil fraction of all the subbase materials was non-plastic.

TABLE 1

SUBBASE MATERIALS

Material	Description
1	Concrete sand with 6% silty loam added
2	Dense-graded sand and gravel
3	Open-graded sand and gravel
4	Dense-graded crushed limestone
5	Open-graded crushed limestone
6	Concrete sand
7	Concrete sand with 12% silty loam added
8	Concrete sand retained on No. 20 sieve
9	Concrete sand passing No. 20 sieve

To assure uniformity, a sufficient quantity of each material was obtained to complete all testing involving that material.

The AASHO moisture-density relationship was used for the control of the placement density in all tests, although for some of the materials the moisture-density curve was not clearly defined. To supplement this method of density control, tests of minimum and maximum density were performed, and the density relationships are also given in terms of relative density. The moisture-density relationships and minimum and maximum densities for each material are shown in Table 2

Triaxial compression tests were performed on each material when compacted to 100 percent of AASHO density at optimum moisture. The values for the angle of internal friction and cohesion are shown in Table 3.

A coefficient of permeability for each subbase material was determined at various densities with a constant head permeameter. These data are shown in Figure 2 as a function of the relative density.

Subgrade

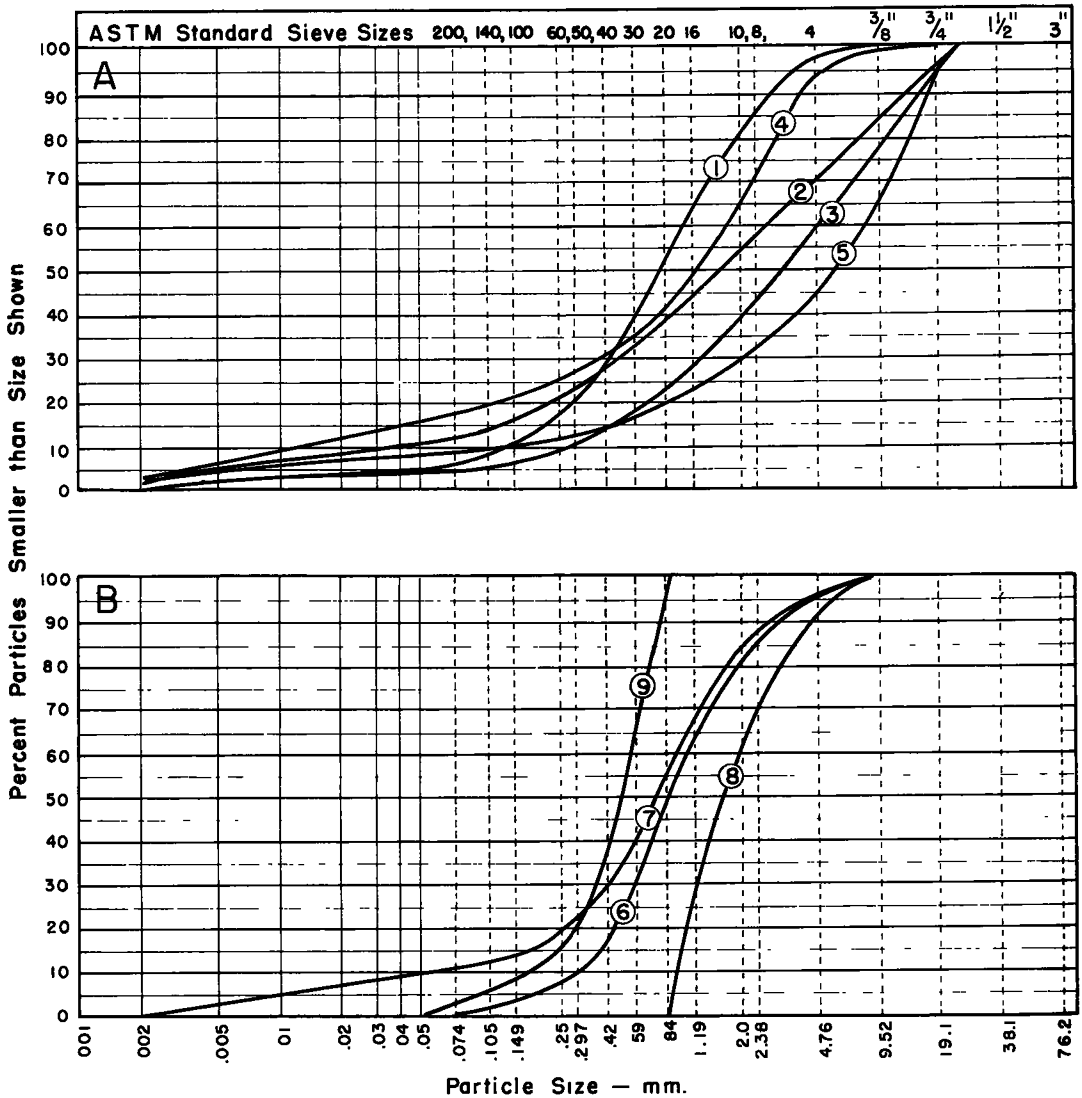
The subgrade material in all tests was a clay soil with the characteristics and gradation shown in Table 2.

TEST EQUIPMENT

The equipment (Figs. 3 through 6) consisted of a container which held the subgrade and subbase materials, a prestressed concrete slab which was loaded and transferred the load to the subbase, a repetitive load system, and instrumentation for measuring deflections, densification and pressures. Five of these test units were used in the investigation.

Soil Container

The soil container was a reinforced concrete box 4 ft wide by 6 ft long by 3 ft deep. The side walls of the box extended 4 in. higher than the end walls to retain a "shoulder surcharge." Unistruts were cast in the sides of the box to act as anchorage for a load



AASHO-ASTM SOIL SEPARATE CLASS

CLAY	SILT	SAND		GRAVEL
		FINE	COARSE	
-0.005	0.005 — 0.074	0.074 - 0.42	0.42 - 2.0	2.0 — 76.2

Figure 1. Grain size accumulation curves.

frame used in performing plate bearing tests on the subgrade and on the subbase. Additional steel was cast in the concrete as anchorage for the repetitive load system.

To provide drainage of the subbase layer when desired, a 2-in. wide, 5-ft long, 7-in. high drain was installed along each side of the soil container at the level of the subbase. The drains, which were fabricated from steel, had a 7-in. open side facing the subbase. This side was covered with a No. 40-mesh sieve placed between two thin steel plates perforated with $^3/_{16}$-in. diameter holes spaced on $^5/_{16}$-in. centers. The drains were placed on a slight slope toward a controlled outlet at one end of the container. When a drain was closed, water could flow from the subbase and accumulate

TABLE 2

SUBBASE AND SUBGRADE CHARACTERISTICS

(a) Subbase

Material[a]	AASHO Moisture-Density Data		Relative Density Data	
	Optimum Moisture, %	Standard Dry Density, pcf	Minimum Density, pcf	Maximum Density, pcf
1	9.0	124	102	135
2	8.0	135	113	149
3	7.5	131	104	144
4	8.5	129	97	147
5	8.0	128	96	142
6	9.0	125	102	135
7	9.5	125	102	133
8	5.0	111	96	114
9	7.0	108	91	113

(b) Subgrade

		Particle Size	%
AASHO Optimum Moisture	20.4%	Sand (2.0-0.74mm)	20
AASHO std. Dry Density	105 pcf	Silt (.074-.005mm)	40
Liquid Limit	48	Clay (below .005)	40
Plasticity Index	24		

[a] All materials non-plastic, except No. 2 which has a PI of 2.

in the drain, when the drain was open, water could flow from the subbase into and out of the drain. Water could also be added to the subbase through the drain.

Concrete Slab

The concrete slab was 2 in. thick, 2 ft wide and 8 ft long; it was separated at mid-length by a ½-in. space to represent a joint in a pavement. The slab was pretensioned, and was cast in place on the subbase. Longitudinal tensioning consisted of 8 equally spaced ¼-in. diameter, 7 strand high strength cables located at the mid-depth of the slab and tensioned to 150,000 psi. The strands extended through the joint. This procedure produced a thin flexible slab capable of withstanding the repetitive loads without

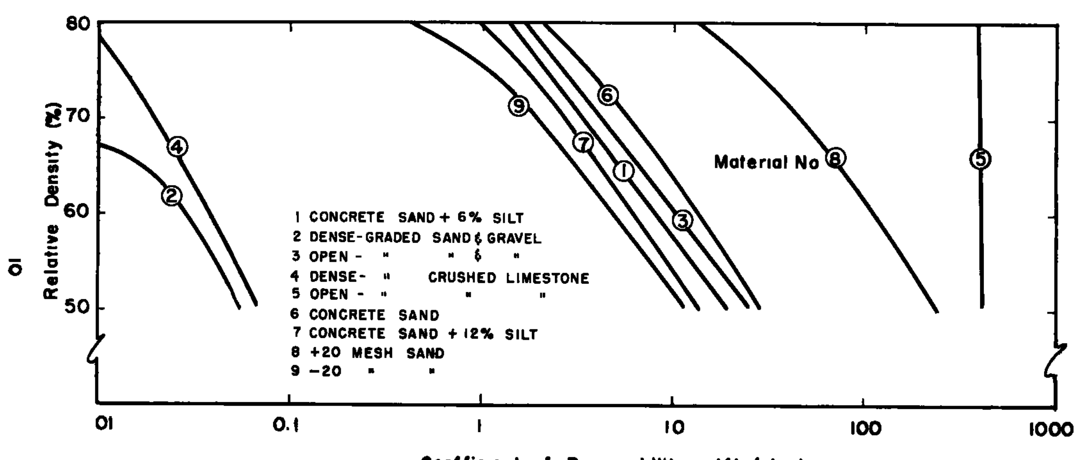


Figure 2. Variation of permeability with relative density.

Figure 3. View of soil container with clay subgrade, subbase drains, and soil pressure cell in place. A plate bearing test is being performed on the subgrade.

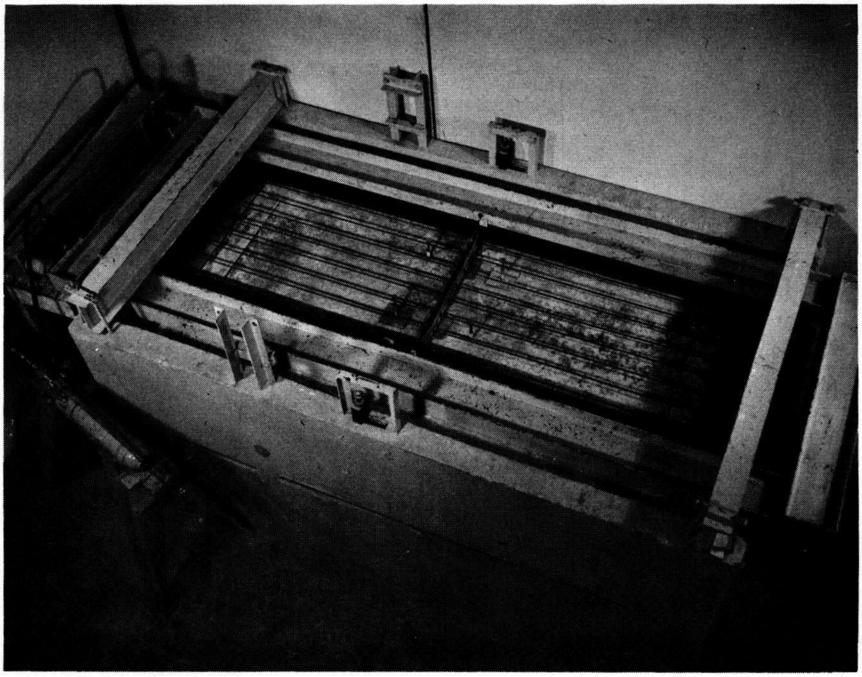


Figure 4. View after placement of subbase material showing prestress frame and wires prior to casting concrete slab. The center-joint dividing strip is in place.

failure. The subbase extended 1 ft on each side of the slab. This area was "shouldered" as described later.

Repetitive Load System

The mechanism for applying repetitive loads consisted of a steel I-beam with ballast weights (Fig. 6). The beam was hinged at one end and was alternately raised and dropped at the loaded end by a Westinghouse air brake. The reaction of the load beam was taken by a thrust rod located about one-fourth of the distance from the hinge to the ballast weights. The thrust rod transmitted the load to an 8-in. diameter steel plate resting on a rubber pad astride the joint in the concrete slab. The slab then transmitted the load to the subbase. The ballast weighed 975 lb, and produced a load of 4,000 lb on the plate which was of sufficient magnitude to transmit a pressure to the subgrade of about 7 psi. This pressure, according to other laboratory tests on full-scale pavements,

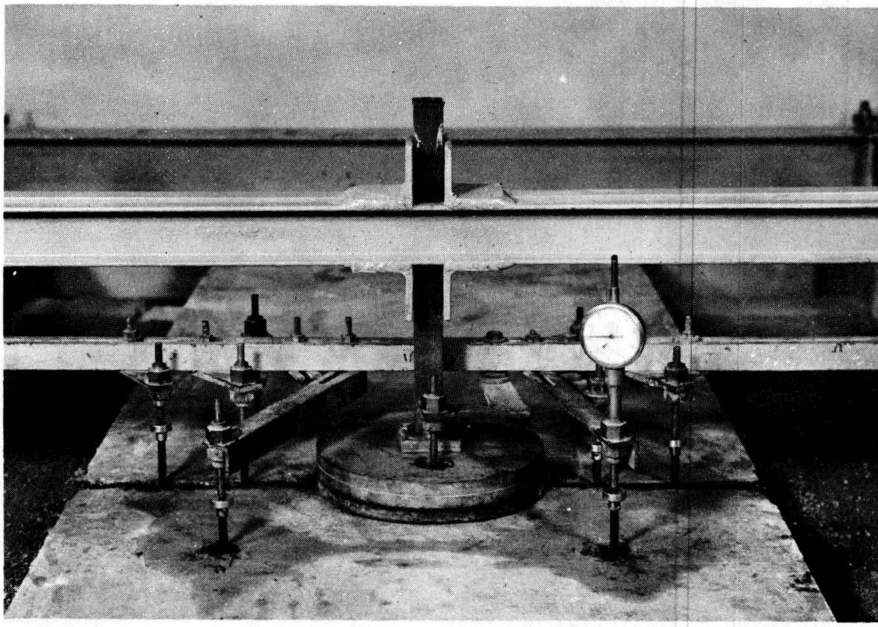


Figure 5. Load plate in position over the joint in the concrete slab. Reference rods and dial gage for obtaining deflection and densification data may be seen.

TABLE 3

TRIAXIAL TEST DATA

Material	Description	Angle of Internal Friction	Cohesion psi
1	Concrete sand plus 6% silt	34	1.0
2	Dense-graded sand and gravel	29	4.0
3	Open-graded sand and gravel	32	0.9
4	Dense-graded crushed limestone	41	5.2
5	Open-graded crushed limestone	42	0.0
6	Concrete sand	36	0.0
7	Concrete sand plus 12% silt	36	2.2
8	Concrete sand retained on No. 20	38	0.3
9	Concrete sand passing No. 20	38	2.2

is equal to or greater than the average pressure that may be expected on the subgrade under a concrete pavement in normal service. The load was applied at a rate of approximately 20 applications per minute. The number of repetitions was measured by a counter attached to the air brake, with the tripping arm fastened to the piston by a spring.

The rate and duration of the load was controlled by solenoid valves activated by an eletric timer. By instrumenting the thrust rod with SR-4 gages the trace shown in Figure 8 was obtained, which illustrates the characteristics of the load used in the test program.

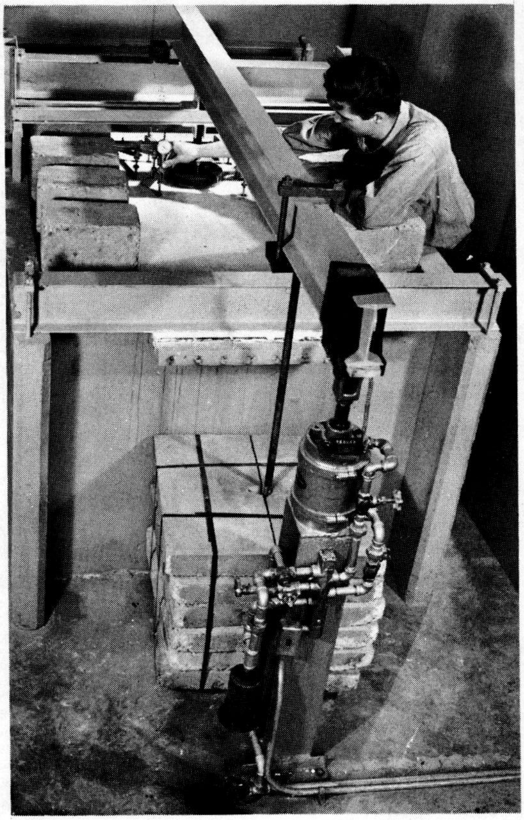


Figure 6. Assembled view of ballast weights, load mechanism and load beam with the shoulder surcharge blocks in position. A densification reading is being obtained on one of the reference rods.

Deflection, Densification and Pressure Devices

Deflection and densification data were obtained using $\frac{1}{4}$-in. steel reference rods attached to $\frac{3}{16}$-in. thick by 3-in. square steel plates. Four devices were installed with the base plates on top of the subgrade and four with the plates on top of the subbase. Two of each type were located at the joint, and two 6 in. away from the joint, (Figs. 5 and 7). Pipe sleeves were used around the rods to prevent them from adhering to the subbase or slab. Finally, one rod was attached to the top of the slab near the joint. The elevations of the top of the rods with respect to a fixed frame were determined with an Ames dial.

These reference rods permitted the following measurements:

1. Deflection of the slab during loading.
2. Decrease in thickness of the subbase layer (densification or consolidation).
3. Lowering of the elevation of the top of the subgrade because of densification (permanent deformation) or because of intrusion of the subgrade into the subbase. (Intrusion, if it occurred, could be observed at the completion of the test when a vertical excavation was made through the subbase into the top portion of the subgrade.)
4. Increasing deflection of the slab due to densification of the subbase and of the subgrade, and due to intrusion of the subgrade into the subbase.
5. Elastic deflection or deformation within the subbase and within the subgrade as distinguished from permanent deformation as measured in items 2 and 3.

To determine the pressure on the subgrade, a Carlson stress meter was installed in each box. The cell was located 6 in. from the joint and was bedded in mortar with the face of the cell level with the top of the subgrade. All cells were calibrated in place.

TEST PROCEDURE

The clay subgrade soil was prepared by crushing the dried material to pass a No. 4 sieve. Water required to bring the soil to optimum moisture content was added and the material was compacted to standard AASHO density in the test box to a depth of 30 in. The soil was compacted in 6-in. layers with a mechanical impact hammer. In-place density tests were made using the sand-cone method. After the density tests,

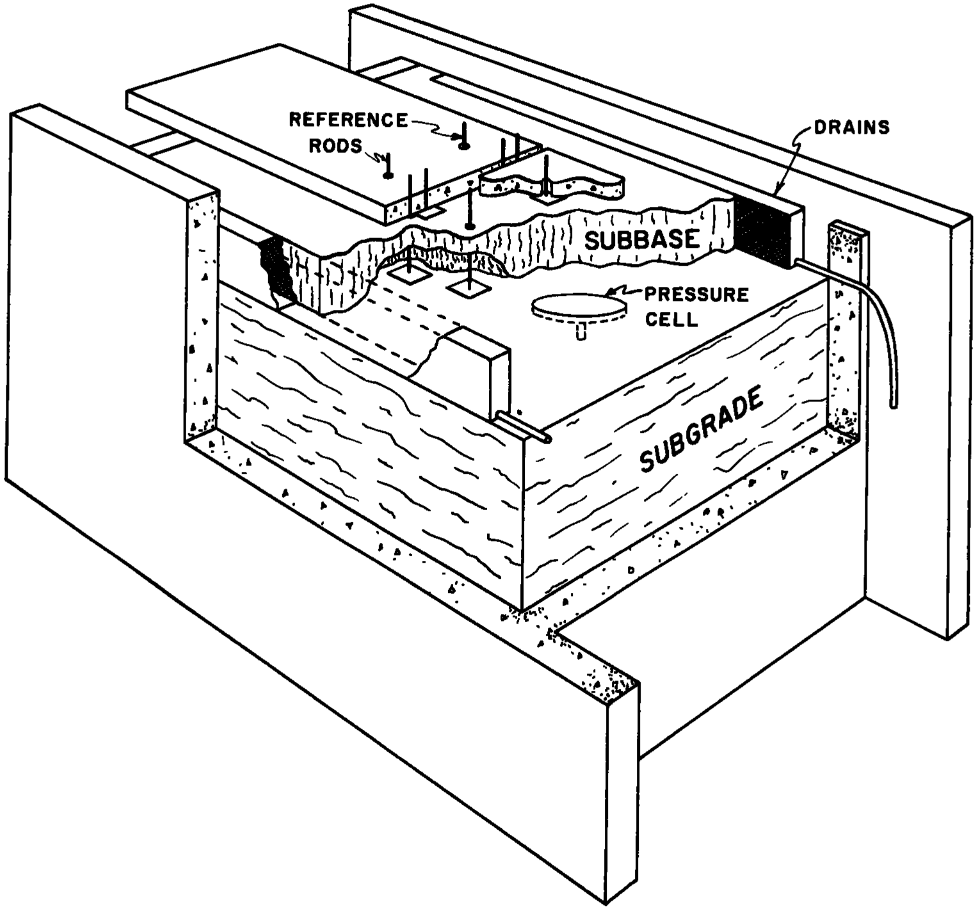
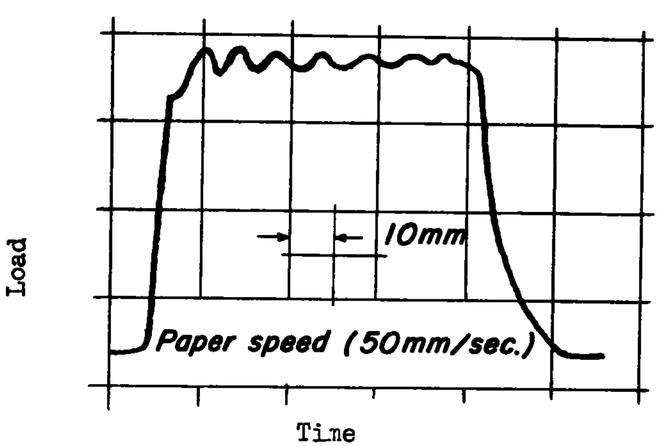


Figure 7. Location of reference rods, pressure cell, and side drains.



Operation	Time (sec.)
Load applied	*0.10*
Load on	*1.38*
Load released	*0.42*
At rest	*1.00*
One cycle	*2.90*

Figure 8. Load characteristics.

the subgrade was leveled, and the pressure cell was installed and calibrated. Plate bearing tests were then made using a 12-in. diameter plate. These tests were repeated at the completion of densification test to determine whether the subgrade "strength" had changed during the repetitive loadings (Table 5).

The design water for subbase placement was mixed with the material in a concrete mixer, and the subbase was then placed at one of several different densities and moisture contents as outlined in Table 4. The material was compacted with a hand tamper to obtain designed densities in the lower range and with a pan vibrator to obtain densities in the higher range. To check compaction control, in-place density determinations were made using the sand-cone method on all materials except the open-graded crushed limestone (material 5) where a 70-30 mixture of plaster of paris and portland cement was used to obtain a cast of the hole. Next, a plate bearing test was made on the subbase with the plate centered over the pressure cell in the subgrade. This procedure provided data relating the influence of density and type of material to the magnitude of pressure transmitted to the subgrade. Reference rods to measure subgrade densification (permanent deformation) were installed on top of the subgrade in the holes made when determining the subbase density. The installation of the reference rods on top of the subbase completed the operation.

TABLE 4

PLAN OF SUBBASE PLACEMENT CONDITIONS

Standard AASHO Density, %	Moisture Content
80	AASHO Standard Optimum
90	AASHO Standard Optimum
100	AASHO Standard Optimum
100	2% below AASHO Standard Optimum
100	2% above AASHO Standard Optimum
110	AASHO Modified Optimum[a]
100 (soil-cement)	AASHO Standard Optimum

[a] Optimum moisture determined by AASHO Modified Moisture-Density Test.

The prestress frame was then set in place, and spacer plates for forming the joint were installed. Cables were tensioned, and the slab was cast. After the slab had cured, the tension in the cables was released, and the prestress frame was removed. Then the loading aparatus was assembled, and initial dial readings obtained. A shoulder surcharge load of 0.7 psi was then placed on the subbase along each edge of the slab. This load was sufficient to retain the subbase material in place without upward shearing movement during the loading operations. The loading apparatus was then set in operation.

To determine the influence of placement density and moisture content on densification, each of the five basic subbase materials was tested under conditions conforming as nearly as practical to the plan shown in Table 4. Materials 6 through 9 and the four soil-cement subbases were tested only at 100 percent of standard dry density and at optimum moisture. The actual density and moisture content of the subbases as placed are shown in Table 5.

The repetitive load test was completed in four stages. During the last three stages water was added to the subbase to simulate extreme conditions of subbase moisture which might be attained in service.

Stage 1—Subbase at Placement Condition

During this stage 150,000 loads were applied to the subbase as placed to determine the amount and rate of densification of the various subbase materials at moisture conditions near the standard or modified optimum and at densities of approximately 80, 90, 100, and 110 percent of standard AASHO density. The drain outlets were open. Depending on the moisture content and permeability of the subbase, some moisture was discharged from the drains.

Stage 2—Subbase Being Wetted

In this stage 150,000 loads were applied to determine the densification characteristics of subbases as the moisture content was increased to saturation. Water was added to the subbase through the joint in the pavement twice daily in a quantity, calculated from the void ratio, sufficient to saturate the subbase during this test stage. The drain outlets were closed but limited drainage from the subbase into the drains could occur.

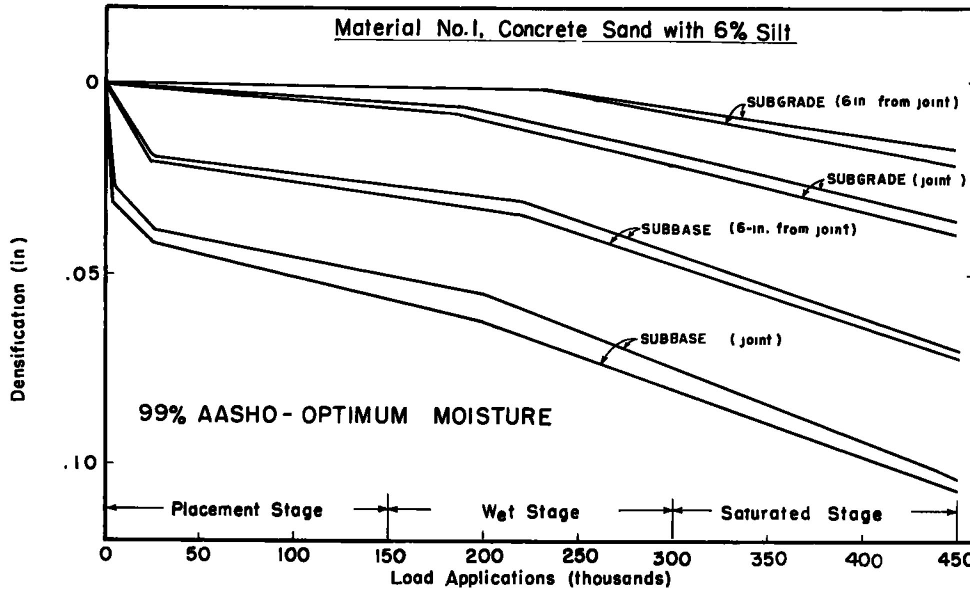


Figure 9. Representative densification data.

Stage 3—Subbase Saturated with No Pumping

In this stage 150,000 loads were applied to determine the densification of a saturated subbase. The drain outlets were kept closed and water was added though the drains to fill them approximately two-thirds full. If during the loading cycles, pumping of the subbase developed, the loading was temporarily stopped until the moisture had disseminated so that a continuation of loading did not result in pumping.

Stage 4—Subbase Saturated with Pumping Permitted

This final stage consisted of 50,000 load repetitions to distinguish between pumping and non-pumping subbases under severe moisture conditions. The drain outlets were kept closed, but the drains were completely filled with water and a visual record was obtained of any pumping which occurred during the loading cycles.

TEST RESULTS

Measurements were made of the permanent deformations associated with decrease in the thickness of the subbase layer and with the lowering of the elevation of the top of the subgrade accompanying an increase in density due to the repeated load applications. For convenience this decrease in thickness of the subbase and the permanent deformation of the subgrade are referred to as densification. This method of reporting was satisfactory in all tests but one in which intrusion of the subgrade into the subbase occurred and as a result, lowering of the elevation of the top of the subgrade was due to a combination of densification and intrusion.

Densification measurements were made periodically at eight locations in each test. The measurements were obtained from the reference rods located on the subbase and on the subgrade, either at the joint or 6 in. from the joint. A complete set of data for illustration is shown in Figure 9. These data are for material number 1 placed at 99 percent of standard AASHO density and at optimum moisture. Good agreement is noted in densification measurements on each pair of reference rods located on either the subbase or the subgrade. The measurements obtained 6 in. from the joint were of secondary significance. When considered for all the tests, they showed that as the subbase placement density was increased, the difference between the densification

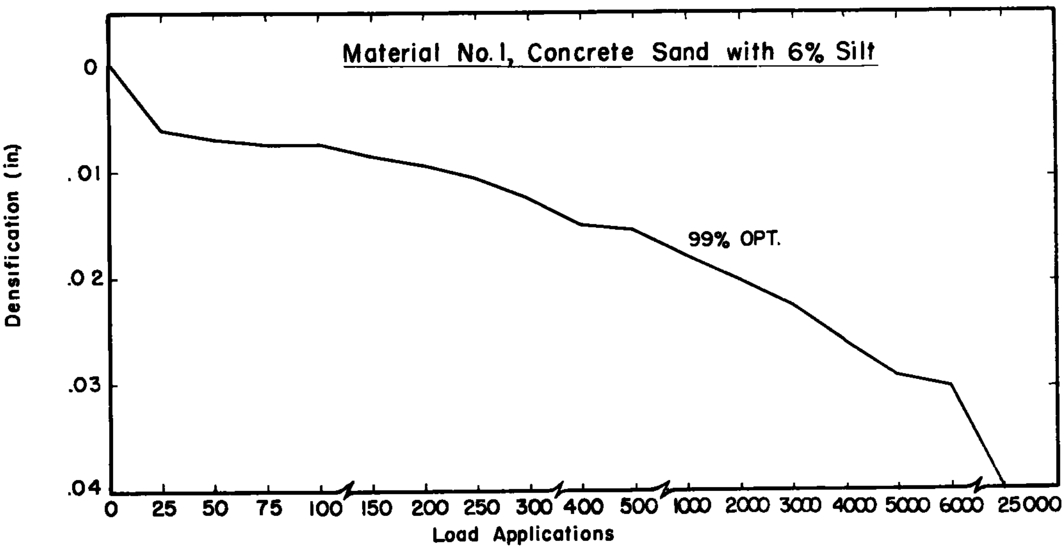


Figure 10. Subbase densification first 25,000 load applications.

at the joint and at 6 in. from the joint became smaller. On the average, the densification 6 in. from the joint varied from 50 to 95 percent of the densification at the joint as the placement density was increased from 80 to 110 percent of standard AASHO density. Only the average densification data measured at the joint are considered in the remainder of the figures.

Replicate tests conducted on each of the subbase materials placed at 100 percent of AASHO density and optimum moisture showed the the greatest discrepancy in the magnitude of densification for any replication was 14 percent.

PERFORMANCE OF GRANULAR SUBBASES

Data indicating the influence of the variables on the densification in the 6-in. subbase layer are shown in Figures 10 through 19. The lettering on each curve defines the placement density as a percent of the standard density, and the initial moisture content as optimum or on the dry or the wet side of optimum as outlined in Table 4. The placement density in pounds per cubic foot, moisture content in percent, and modulus of soil reaction in pounds per cubic inch are shown for each test in Table 5.

The influence of the variables on densification is discussed by considering the performance of the various subbase materials during the four stages of loading.

Placement Stage

During the first 150,000 load applications (placement stage) the drains were open and no water was added to the materials; however, water could flow from the materials into the drains. Densification during this stage varied from an almost negligible amount (0.025 in.) to more than 0.25 in. The variable which was most significant was the placement density, although moisture content and gradation were also significant.

About 60 percent of the densification occurred during the first few thousand applications of load. This rapid rate of densification is illustrated in Figure 10 by an expanded plot of the early portion of loading for the data of Figure 9. By comparing Figures 9 and 10 it may be seen that about 10 percent of the densification obtained after 150,000 load applications had occurred after only 25 load applications, and about 50 percent had occurred after 6,000 load applications.

Generally, about 80 percent of the densification that occurred during the placement stage was obtained after 25,000 load applications. This was observed with all of the materials and for all of the conditions of placement density and placement moisture. The magnitude of densification during this period varied from 0.022 in. to 0.18 in.

TABLE 5

SUMMARY OF PLACEMENT DATA

Initial Subbase Moist. and Density		Subbase Init. % Standard AASHO Den.	Subbase Relative Density Initial	Subbase k (12-in. plate)		Initial Subgrade Moist. and Density		Subgrade k (12-in plate)	
W.[a]	Den.[b]			Init.	Final	W.	Den.	Init.	Final
				Material 1					
8.6	103.0	83	46	175	490	20.5	106.0	310	255
8.6	116.4	94	51	320	420	21.8	110.0	350	340
8.3	122.4	99	68	365	485	22.9	103.0	340	285
12.0	123.8	100	72	290	418	20.2	110.0	355	365
6.5	122.7	99	68	340	500	20.5	106.5	394	410
7.3	133.6	108	97	504	510	18.9	109.8	305	305
8.3[c]	120.0	97	62	332	516	21.6	106.0	300	306
8.0[d]	123.2	100	72	271	455	19.0	108.0	260	297
				Material 2					
7.3	104.1	79	-	87	278	19.4	106.0	339	308
7.8	120.9	90	27	299	394	22.3	103.5	305	278
7.8	136.9	102	72	420	673	20.0	103.6	365	353
9.9	136.3	101	70	406	696	21.0	106.7	339	411
6.1	133.9	100	65	390	520	22.8	107.4	285	305
5.2	143.6	107	88	418	522	23.2	104.0	278	288
				Material 3					
7.2	105.1	80	34	18	-	19.2	102.6	291	309
7.4	117.7	90	40	217	430	21.5	109.4	388	364
7.0	132.7	101	78	290	338	21.4	102.7	408	500
9.6	133.7	102	80	370	408	20.3	106.5	400	455
5.7	128.7	98	69	404	503	20.6	103.1	296	308
6.0	143.0	109	98	508	700	22.2	108.3	364	388
				Material 4					
9.0	105.9	82	25	280	432	21.2	106.0	348	312
8.7	115.4	90	47	300	427	19.5	101.4	425	384
8.8	127.9	99	71	424	464	20.2	105.2	309	341
10.8	129.0	100	73	437	682	19.5	105.0	344	325
7.2	129.4	100	73	390	666	20.8	109.7	306	319
6.2	139.5	108	90	475	850	19.6	103.9	387	336
				Material 5					
8.2	104.0	81	24	232	357	19.5	104.9	279	245
8.6	113.2	88	47	383	529	20.0	107.9	328	304
8.3	128.2	100	78	386	650	21.0	101.0	393	414
11.0	129.7	101	80	312	465	22.1	100.2	381	377
6.1	130.4	102	81	464	394	23.6	105.0	307	312
7.8	140.1	109	97	531	610	22.0	102.9	305	280
				Material 6					
7.8	121.0	97	71	334	482	18.8	110.5	280	257
				Material 7					
9.1	126.5	101	83	420	510	18.0	113.0	305	279
				Material 8					
6.8	106.8	97	65	348	371	19.2	106.9	366	383
				Material 9					
6.8	109.5	101	85	383	460	19.6	112.3	274	279

[a] Moisture content, percent by dry weight of material
[b] Density, pounds per cubic foot, dry.
[c] 12-in. thickness
[d] 4-in. thickness.

To facilitate comparing the materials at the various placement conditions, densifications at the end of 25,000 load applications and at the completion of the placement stage (150,000 applications) are given in Table 6. The benefit of increased subbase placement density in reducing the magnitude of densification under repetitive loading may be observed in this table. For example, based on the average densification data for the basic subbase materials, an increase in placement density from 80 percent to 90 percent of standard reduced the densification by 63 percent; the increase from 90 to 100 percent resulted in a reduction of 41 percent; and, the increase from 100 to 110 percent resulted in a reduction of 20 percent.

If a densification of about 0.05 in. for this test condition is arbitrarily selected as a basis for designating superior performance, it is found that materials 1 through 5 and material 7 meet this criterion when compacted to 100 percent or more of AASHO standard density at optimum moisture. Densifications greater than 0.05 in. were shown by materials 6, 9 and 8, increasing in that order. However, after the rapid rate of densification during the first 25,000 load applications, each of the materials showed only slight additional densification (0.01 to 0.03 in.) during the remaining 125,000 load applications, and each could be considered as showing satisfactory performance at the completion of the first stage. This conforms to the concept that satisfactory subbase performance can be obtained with most properly compacted granular materials provided the moisture content can be maintained near the optimum.

The effect of drier placement (2 percent less than optimum) as determined for materials 1 through 5 at 100 percent density varied with the type of material, but in general the densification was about the same as when the subbase was placed at optimum.

TABLE 6

DENSIFICATION (INCHES) OF 6-IN: SUBBASES AT END OF 25,000 AND 150,000 APPLICATIONS

	Placement Conditions											
Approximate Density (% Standard)	80		90		100		100		100		110	
Moisture	opt.		opt.		opt.		opt. -2%		opt. +2%		mod. opt.	
Load Applications (thousands)	0 to 25	0 to 150	0 to 25	0 to 150	0 to 25	0 to 150	0 to 25	0 to 150	0 to 25	0 to 150	0 to 25	0 to 150
1 Concrete Sand, plus 6% silt	0.180	0.215	0.043	0.053	0.040	0.050	0.030	0.045	0.075	0.090	0.022	0.032
2 Dense Graded Sand and Gravel	0.257	0.280	0.073	0.085	0.033	0.050	0.045	0.045	0.048	0.060	0.027	0.043
3 Open Graded Sand and Gravel	0.140	0.147	0.067	0.067	0.025	0.032	0.030	0.040	0.060	0.070	0.020	0.027
4 Dense Graded Crushed Limestone	0.045[a]	0.060[a]	0.032	0.075	0.032	0.042	0.035	0.042	0.050	0.060	0.025	0.036
5 Open Graded Crushed Limestone	0.105	0.125	0.047	0.075	0.025	0.030	0.030	0.060	0.053	0.100	0.017	0.027
Average Densification (Basic Materials 1-5)	0.170	0.194	0.052	0.071	0.031	0.041	0.034	0.046	0.057	0.072	0.022	0.033
6 Concrete Sand	-	-	-	-	0.055	0.080	-	-	-	-	-	-
7 Concrete Sand plus 12% Silt	-	-	-	-	0.030	0.030	-	-	-	-	-	-
8 Concrete Sand, retained on No. 20 sieve	-	-	-	-	0.062	0.100	-	-	-	-	-	-
9 Concrete Sand passing No. 20 sieve	-	-	-	-	0.062	0.085	-	-	-	-	-	-

[a] Anomaly in data, not included in average.

An increase in placement moisture to 2 percent above optimum showed an adverse effect in all cases, and for this condition the densification of all materials exceeded the 0.05 in. criterion. The effect of the increased placement moisture was more pronounced for materials 1, 3 and 5, which were open-graded, than for the dense-graded materials 2 and 4. This appears reasonable, as water could drain from the open-graded materials, thus permitting additional densification under repetitive loading. The dense-graded materials could not drain as rapidly, and densification was thus delayed. This may explain why a placement moisture increase with the open-graded materials was in some cases more adverse than was a 10 percent reduction in placement density.

The above discussion pertains only to the placement stage of the testing, when no moisture was added to the subbase. For the majority of service conditions water would be available to the subbase, and the discussion of the test data in the later stages is considered more pertinent to the problem of subbase densification than the placement stage data.

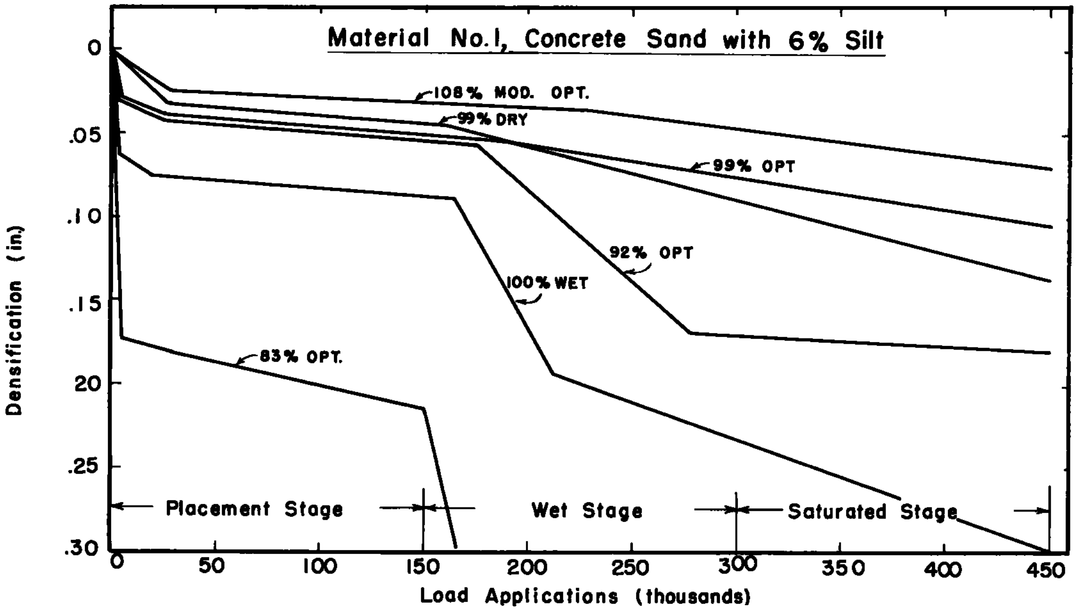


Figure 11. Influence of placement conditions on subbase densification.

Wet Stage

During the wet stage, 150,000 applications of load were applied to determine the densification characteristics of subbases as the moisture content was gradually increased to saturation. Water was added to the subbase twice daily in a quantity calculated to saturate the subbase by the end of the wet stage.

The magnitude of densification that occurred during the wet stage and the total densification at the completion of the wet stage are given in Table 7. This table does not include data for the subbases placed at 80 percent of standard density, as in several cases the densification became so large that the slabs broke and the tests were discontinued.

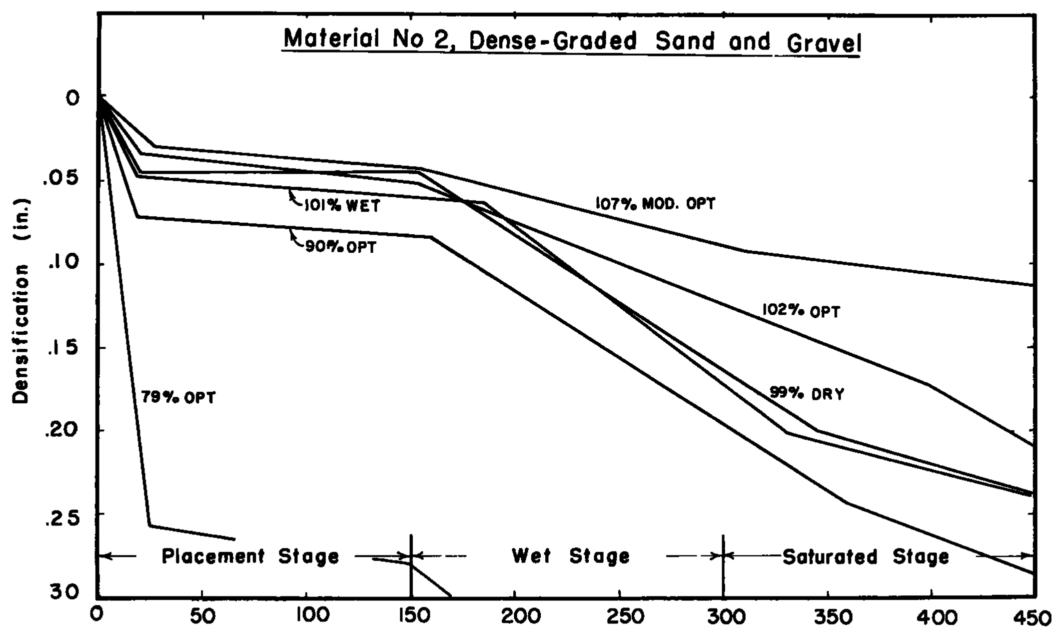


Figure 12. Influence of placement conditions on subbase densification.

The addition of water increased the rate and magnitude of densification for each material at all of the placement conditions. The increase in densification and the time required after the water was added for the detrimental effects to become apparent (as indicated by the first change in the rate of densification after 150,000 load applications) were functions of the test variables. As in the placement stage of loading, the most

TABLE 7

DENSIFICATION (INCHES) OF 6-IN. SUBBASES DURING WET STAGE AND AT END OF 300,000 APPLICATIONS

	Placement Conditions									
Approximate Density (% Standard)	90		100		100		100		110	
Moisture	opt.		opt.		opt. -2%		opt. +2%		mod. opt.	
Load Applications (thousands)	150 to 300	0 to 300	150 to 300	0 to 300	150 to 300	0 to 300	150 to 300	0 to 300	150 to 300	0 to 300
1 Concrete Sand plus 6% silt	0.119	0.172	0.025	0.075	0.045	0.090	0.145	0.235	0.010	0.042
2 Dense Graded Sand and Gravel	0.112	0.197	0.075	0.125	0.117	0.162	0.112	0.172	0.047	0.090
3 Open Graded Sand and Gravel	0.093	0.160	0.043	0.075	0.040	0.080	0.100	0.170	0.018	0.045
4 Dense Graded Crushed Limestone	0.092	0.167	0.010	0.052	0.029	0.071	0.045	0.105	0.011	0.047
5 Open Graded Crushed Limestone	0.034	0.109	0.030	0.060	0.020	0.080	0.025	0.125	0.013	0.040
Average Densification (Basic Materials 1-5)	0.090	0.161	0.037	0.077	0.050	0.097	0.085	0.161	0.021	0.056
6 Concrete Sand	-	-	0.015	0.095	-	-	-	-	-	-
7 Concrete Sand plus 12% silt	-	-	0.021	0.051	-	-	-	-	-	-
8 Concrete Sand retained on No. 20 sieve.	-	-	0.035	0.135	-	-	-	-	-	-
9 Concrete Sand passing No. 20 sieve	-	-	0.015	0.100	-	-	-	-	-	-

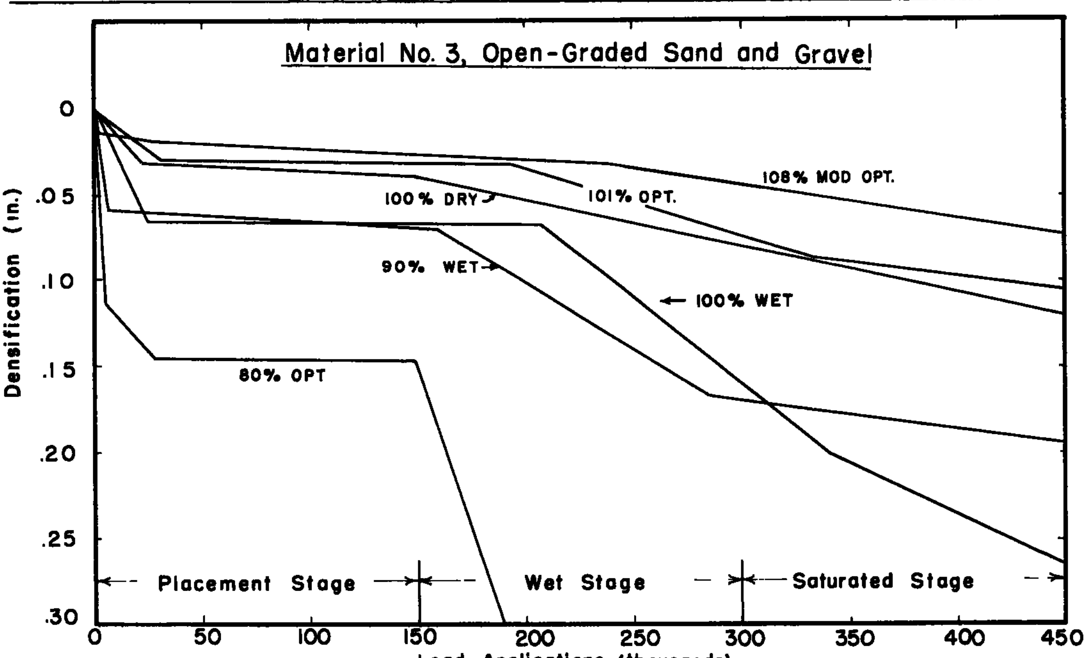


Figure 13. Influence of placement conditions on subbase densifiction.

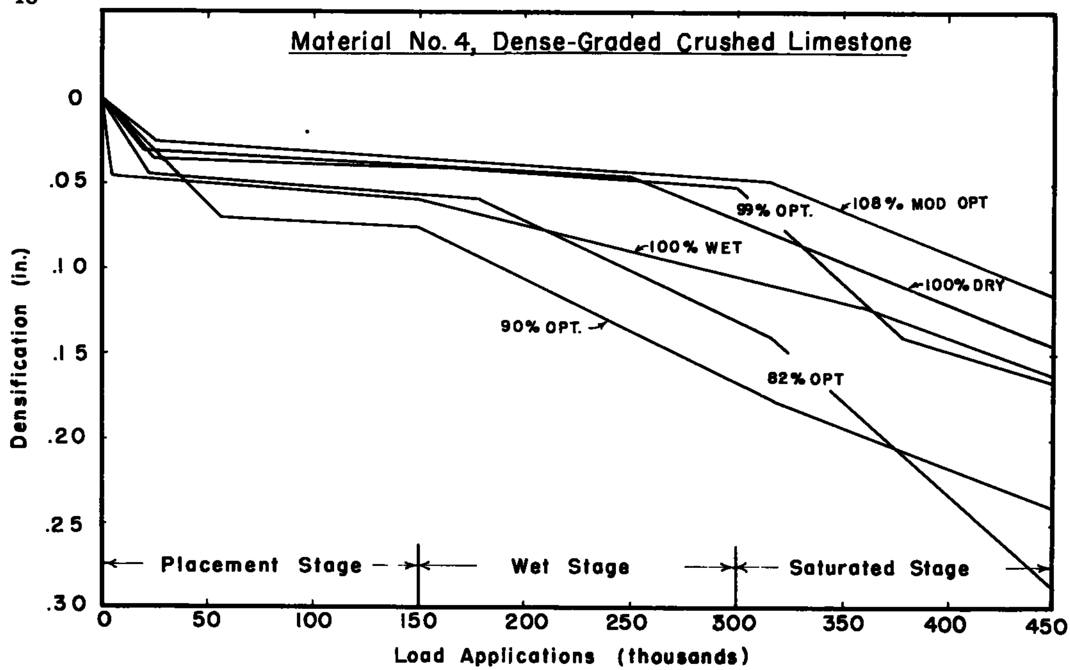


Figure 14. Influence of placement conditions on subbase densification.

significant variable in the wet stage was the placement density, although placement moisture and gradation of the subbase were also significant.

The data show that materials compacted at the higher placement densities were the least influenced and were the last to show the effects of the added water. For example, at a placement density of about 110 percent of standard, the average densification during the wet stage was 0.021 in., whereas at a placement density of 90 percent of standard the average densification was 0.090 in. Furthermore (as can be determined from Figs. 11 to 15) the average number of load repetitions at which the effects of the added water first became apparent was 240,000 for the materials placed at 110 percent density and 170,000 for the materials placed at 90 percent density.

In this stage the materials placed wet of optimum continued to show a greater densification than the materials placed at optimum or dry of optimum. Generally, the materials placed dry of optimum showed the adverse influence of the added water before the materials placed at optimum . Furthermore, the densification of the materials placed dry of optimum was greater than that of the materials at optimum.

Of the five basic subbase materials, dense-graded material 2 (Fig. 12) was the first to show the adverse influence of the added water (average 162,000 load applications) and showed a greater densification in this stage than any of the other materials. In contrast, dense-graded material 4 was not influenced by the addition of water in this stage. This material showed only slight densification and the rate of densification did not change between 25,000 and 300,000 load applications. A possible explanation for the difference in behavior between the two dense-graded materials may be found in the type of soil binder. The binder of material 2 was a plastic clay which could be influenced rapidly by the added water; whereas the binder of material 4 was a non-plastic limestone dust which would not be influenced so rapidly. The rate of densification of the open-graded materials during the wet stage was variable, but tended to increase when the materials were compacted at densities below standard and at moisture contents above optimum.

In considering the sandy soils of varied gradation, good performance (Fig. 16) was achieved during the wet stage with all materials except material 8.

In general, the increase in the rate of densification during the wet stage of loading was greater than that during the "placement stage" and provided a more distinct separation between the materials with respect to performance at the various placement

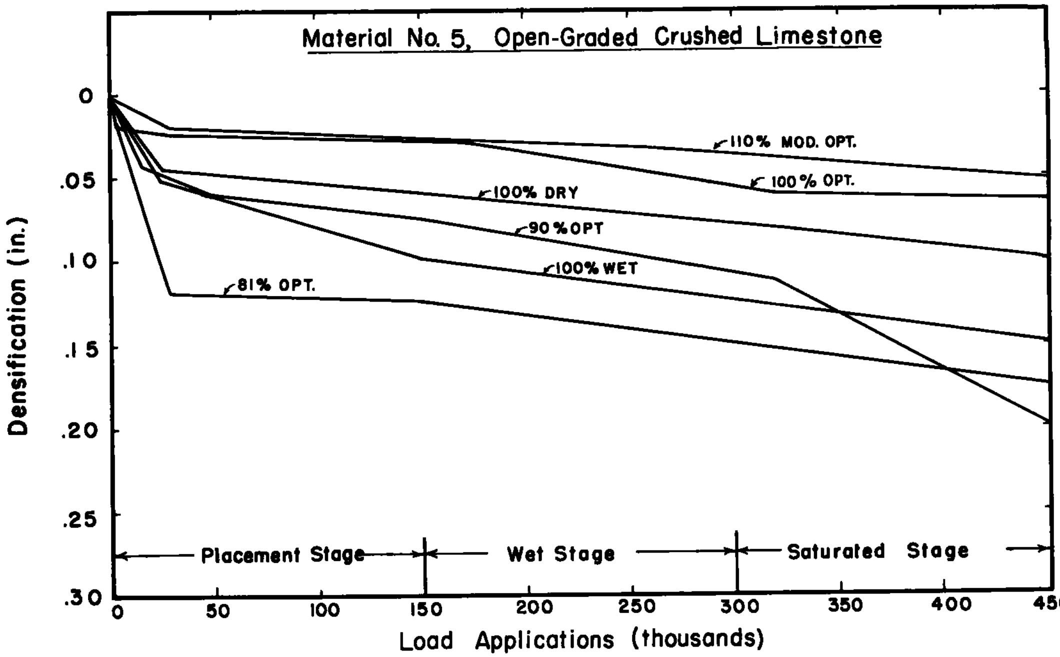


Figure 15. Influence of placement conditions on subbase densification.

conditions. In the saturation stage the separation is further amplified, and the influence of the type or gradation of the material is of greater significance at the completion of the saturation stage.

Saturation Stage

In this stage 150,000 loads were applied to determine the performance of the saturated subbases. Water was added to the drains to fill them approximately two-thirds full. This provided a water reservoir which maintained the subbase in a saturated

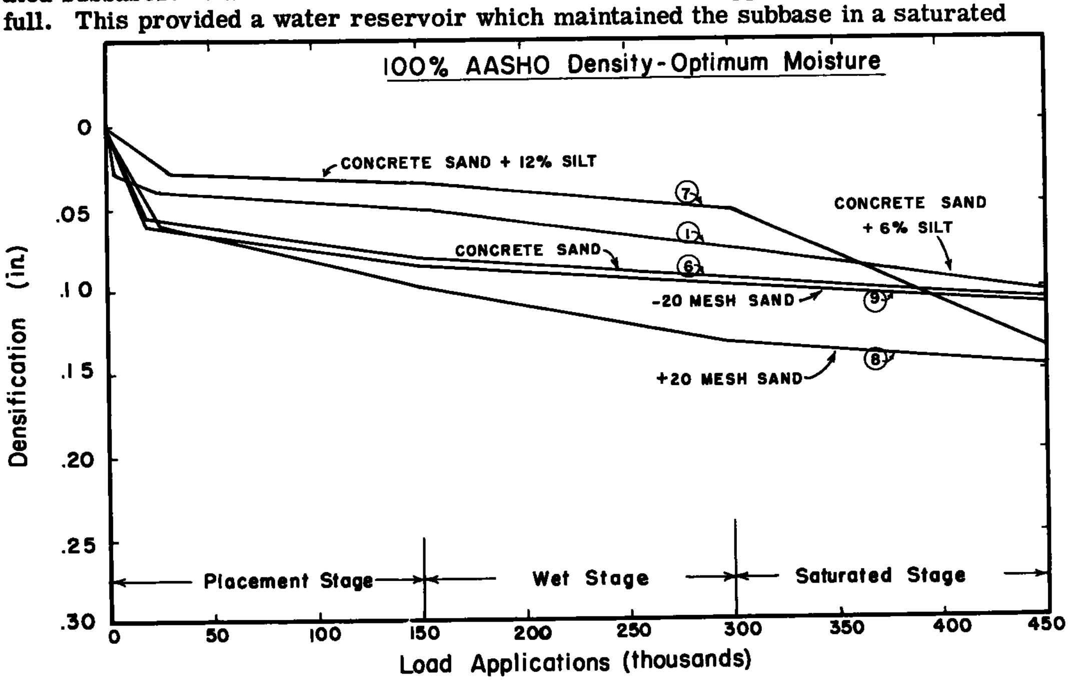


Figure 16. Influence of gradation on subbase densification.

TABLE 8

DENSIFICATION (INCHES) OF 6-IN. SUBBASES DURING SATURATED STAGE AND AT END OF 450,000 APPLICATIONS

	Placement Conditions									
Approximate Density (% Standard)	90		100		100		100		110	
Moisture	opt.		opt.		opt. -2%		opt. +2%		mod. opt.	
Load Applications (thousands)	300 to 450	0 to 450	300 to 450	0 to 450	300 to 450	0 to 450	300 to 450	0 to 450	300 to 450	0 to 450
1 Concrete Sand plus 6% silt	0.008	0.180	0.030	0.105	0.045	0.135	0.065	0.300	0.028	0.070
2 Dense Graded Sand and Gravel	0.078	0.275	0.085	0.210	0.075	0.237	0.068	0.240	0 023	0.113
3 Open Graded Sand and Gravel	0.105	0.265	0.031	0.106	0.040	0.120	0.025	0.195	0.028	0.073
4 Dense Graded Crushed Limestone	0.073	0.240	0.111	0.163	0.069	0.140	0.052	0.157	0.063	0.110
5 Open Graded Crushed Limestone	0.081	0.200	0.005	0.065	0.020	0.100	0.025	0.150	0.013	0.053
Average Densification (Basic Materials 1-5)	0.068	0.232	0.052	0.129	0.050	0.146	0.047	0.208	0 031	0.084
6 Concrete Sand	-	-	0.015	0.110	-	-	-	-	-	-
7 Concrete Sand plus 12% silt	-	-	0.084	0.135	-	-	-	-	-	-
8 Concrete Sand retained on No. 20 sieve	-	-	0.015	0.150	-	-	-	-	-	-
9 Concrete Sand passing No. 20 sieve	-	-	0.015	0.115	-	-	-	-	-	-

state. If pumping developed in this stage (this occurred to a limited degree only with materials 2 and 4), the loading was temporarily stopped until the moisture had disseminated so that a continuation of loading did not result in pumping. Densification during this stage varied from about 0.01 in. to more than 0.10 in. The placement density was still the most significant variable although the type of material was also of considerable significance.

The densifications that occurred during the saturation stage and the total densification at the completion of the saturation stage are given in Table 8. These data show that for this stage the increase in placement density from 90 to 100 percent of standard resulted in a 56 percent reduction in the magnitude of densification. This is about the same percentage of reduction in densification that occurred for similar density increases in the placement and wet stages of loading.

The influence of placement moisture on the magnitude of densification that occurred during the saturation stage was not as significant as it was during the placement and wet stages. This is not surprising as the effects of variation in placement moisture have been equalized by the stage of saturation.

The influence of the type or gradation of the material on the densification during the saturation stage was most apparent with materials 4 and 7. These materials, which had not shown the adverse influence of the added water during the wet stage, showed a rapid increase in the rate of densification during the saturation stage. In contrast, the other materials continued to show about the same rate of densification that had been observed in the wet stage. Slight pumping developed with dense-graded materials 2 and 4 late in the saturation stage, and it was necessary to suspend loading operations to allow time for the moisture to disseminate. However, no pumping was observed with any of the open-graded materials.

In Figure 17a the densification is shown for each of the basic subbase materials placed at approximately 100 percent of standard density and optimum moisture. A comparison of Figures 16 and 17a shows that at the end of 450,000 applications of load the smallest magnitude of subbase densification occurred with open-graded material 5, and the greatest densification occurred with dense-graded materials 2 and 4. In

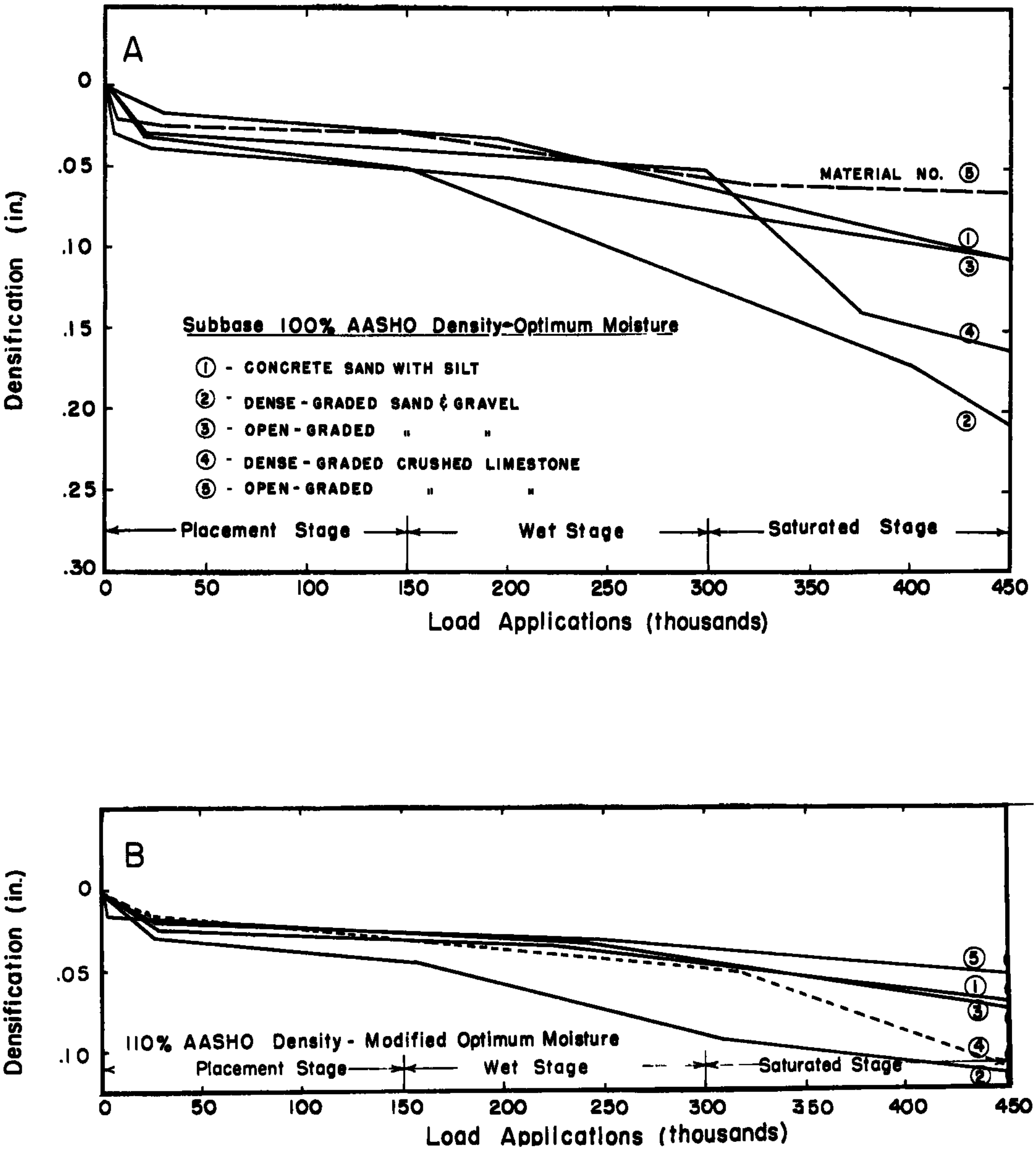


Figure 17. Influence of type of subbase material on densification.

general, the magnitude of densification of the subbase materials was related to the permeability and percentage of material passing the 200 sieve. As the permeability decreased and the percentage of minus 200 increased, the magnitude of subbase densification increased. However, as shown in Figure 17b an increase in the placement density of materials 2 and 4 from 100 to 110 percent of standard reduced the densification to about the same magnitude as that of the open-graded materials placed at 100 percent of standard.

Pumping Stage

During the pumping stage the drains were completely filled with water and 50,000 load applications were applied to the saturated subbase. No densification measurements were obtained and if pumping developed it was allowed to proceed without stopping the load operations.

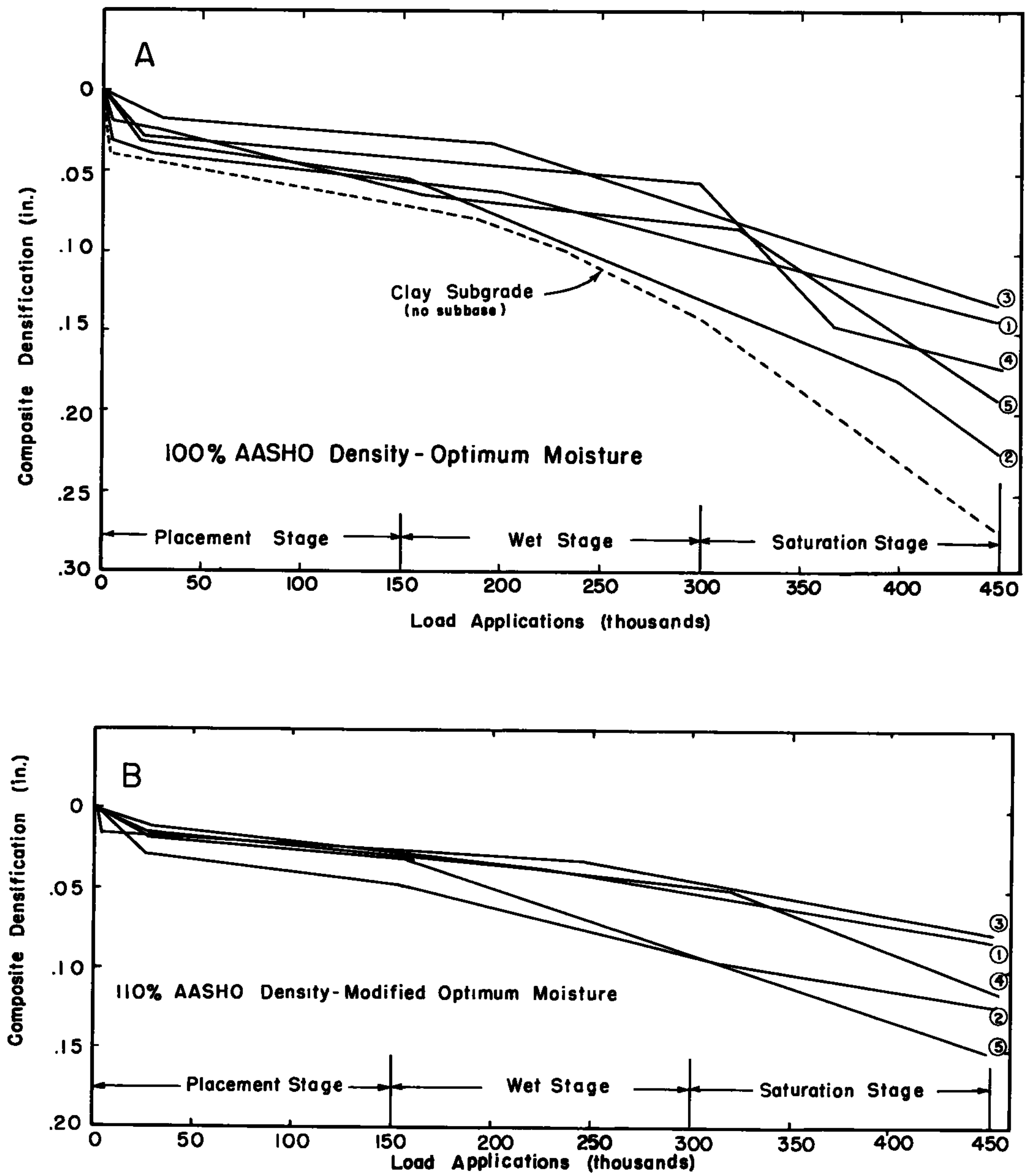


Figure 18. Influence of type of subbase material on composite densification.

No pumping was observed with the subbase materials numbers 1, 3, 5, 6, 8 and 9. In these materials the percentage of minus 200 ranged from 0 to 8 percent and the permeability ranged from 500 ft per day to 3 ft per day. This is significant in view of the pumping that occurred with materials 2 and 4, with the permeabilities of about 0.01 ft per day and 0.03 ft per day, and minus 200 contents of 12 and 17 percent, respectively. The pumping that occurred with material 4 was less severe than the pumping with material 2. Perhaps this was due to the non-plasticity of the soil binder in material 4. Slight indication of pumping of material 7 was observed, although this material had a permeability of about 5 ft per day; however, the minus 200 was 12 percent. The pumping of materials 2 and 4 under adverse conditions was objectionable and emphasized that for conditions of heavy saturation and frequent heavy loads, considerable care is necessary in selecting the type of subbase to prevent pumping and minimize densification. The data showed that no pumping occurred when the subbase material contained less than about 10 percent minus 200 and that densification was the least for well-graded materials having high permeability. However, the gradation must not be so coarse that intrusion of the subgrade may occur. This will be obvious in the discussion to follow on composite subbase-subgrade densification.

Composite Subbase-Subgrade Densification and Performance

Figure 18a shows the densification of the composite subbase-subgrade foundation for each of the basic subbase materials placed at approximately 100 percent of standard density and at optimum moisture, and also data indicating the "densification" of a clay subgrade when a slab was cast directly on the soil (no subbase). Pumping, beyond control, developed in this test soon after the addition of water in the "wet stage" and continued throughout the test. Thus, in this case most of the so-called densification actually consisted of loss of soil due to pumping. A comparison of the test data in Figure 18a with the data on subbase densification in Figure 17a shows that the subgrade densification or permanent deformation under subbase materials 4, 2, 1 and 3 ranged from 0.01 in. to about 0.04 in. This indicates that with these subbase materials which had permeabilities ranging from about 0.01 ft per day to 9 ft per day, there was little or no softening of the subgrade. This was further confirmed by the almost elastic behavior of the subgrade after the first several thousand load applications. However, with material 5, which had a permeability of 500 ft per day, the subgrade softened during the wet and saturation stages and there was severe intrusion of the subgrade into the subbase, This was readily apparent when the sample was removed after test. Principally as a result of this intrusion, the elevation of the top of the subgrade under subbase material 5 lowered 0.13 in. This resulted in a total settlement or measured "composite densification" of 0.2 in., which is greater than the composite densification of dense-graded material 4, but less than that of material 2. Further evidence of the adverse effect of the subgrade intrusion on the over-all performance of material 5 may be seen in Figure 18b for subbases compacted at 110 percent AASHO density.

One method of preventing intrusion of the subgrade and the eventual development of

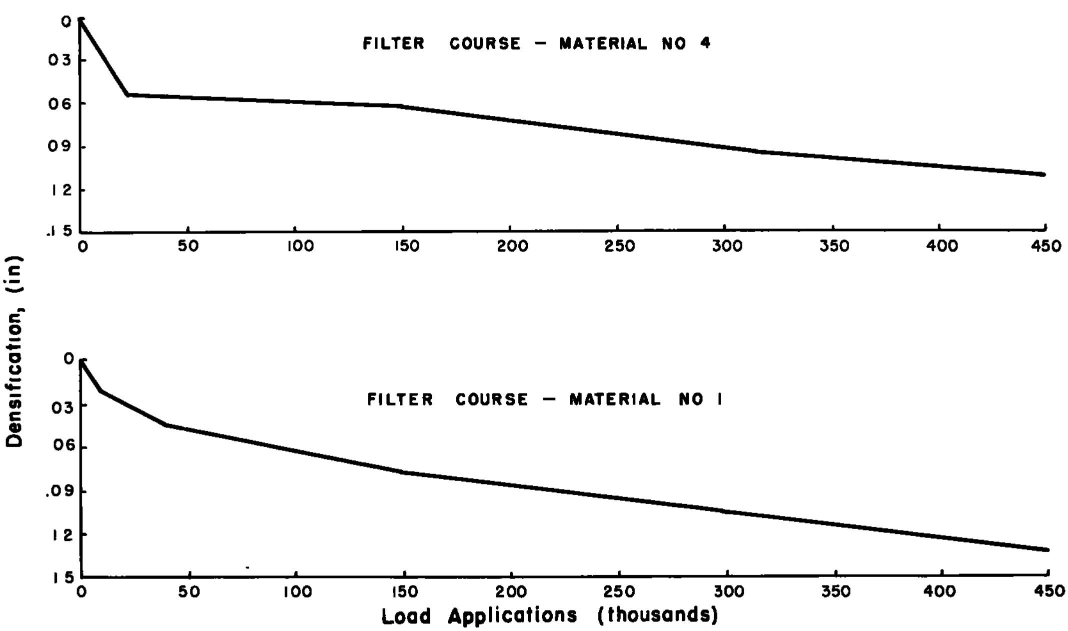


Figure 19. Composite densification of material No. 5 with filter course.

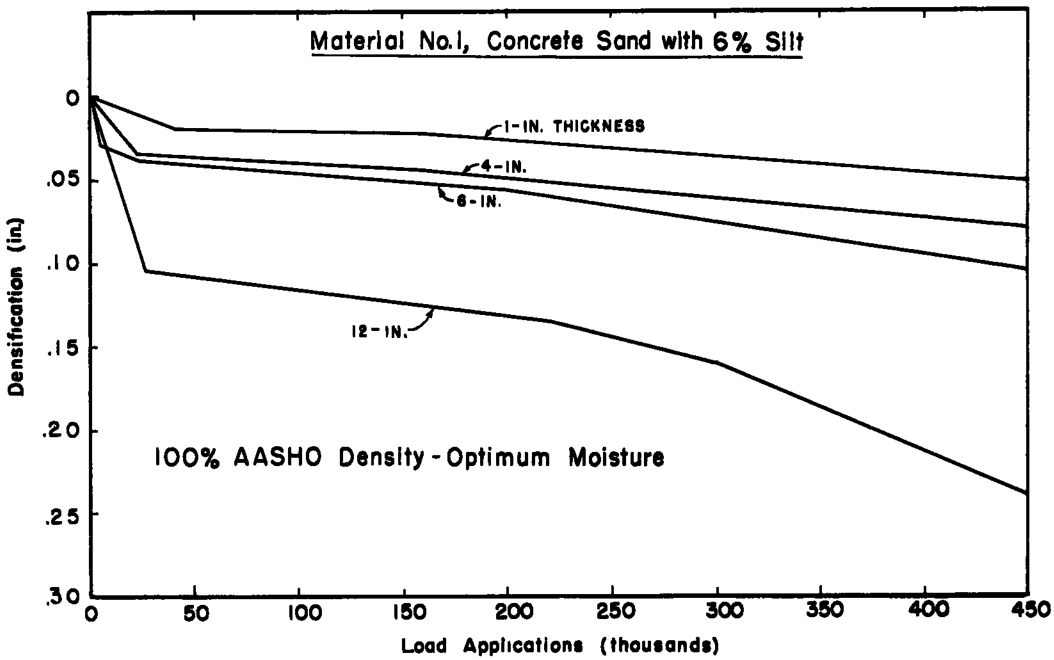


Figure 20. Influence of subbase thickness on densification.

subgrade pumping through the open-graded subbase is to place a filter course between the subgrade and the open-graded subbase. Several criteria have been proposed for the gradation of filter courses. The Corps of Engineers, U.S. Army (3) has established one criterion which states that satisfactory performance may be obtained if the particle sizes of the filter and protected material have the following relationship:

$$\frac{D_{15}\text{ filter material}}{D_{85}\text{ protected material}} \leq 5$$

where

D = particle size, such that

15 percent of filter particles are smaller than D_{15}

85 percent of soil particles are smaller than D_{85}

Materials 1 and 4 were selected for use as a filter course because they met the gradation requirements and furthermore represented both a pumping and non-pumping type of subbase. The use of a 1-in. layer of these materials under a 5-in. layer of material 5 completely prevented subgrade intrusion. This is shown in Figure 19 where composite densification is 0.13 in. or less. This compares favorably with the composite densification for any of the other subbase-subgrade combinations shown in Figure 18a.

Influence of Subbase Thickness on Densification and Pumping

The influence of the thickness of the subbase layer on the magnitude of subbase densification is shown in Figure 20. The data show that the magnitude of densification increased with an increase in the thickness of the subbase layer. With the exception of the 1-in. thickness, the magnitude of densification was approximately proportional to the thickness of the layer.

It is significant that even a 1-in. thickness of subbase material 1 prevented pumping. However, with only 1 in. of subbase the magnitude of subgrade densification or permanent deformation was about 0.12 in. The 4-in. subbase reduced the subgrade densification to about 0.04 in., and a 12-in. subbase reduced the amount to 0.03 in. The smallest magnitude of composite subbase-subgrade densification was obtained with the 4-in. subbase. Thus, with some subbase materials it would appear unwarranted and even undesirable to construct a subbase with a thickness greater than that required to minimize the composite subbase subgrade densification and to prevent pumping. It is apparent that to fulfill these objectives the subbase material should be graded as a filter course that would prevent infiltration of the subgrade into the subbase.

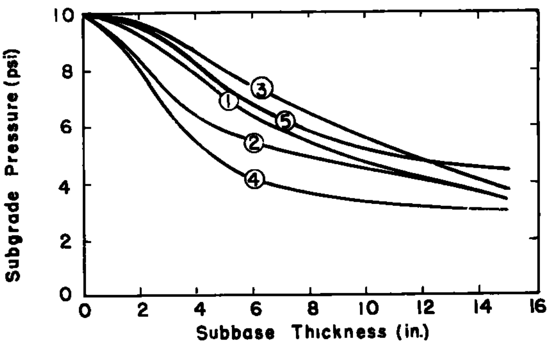


Figure 21. Pressure transmitted to subgrade through various types and thicknesses of subbase. (10 psi load on 12-in. dia. plate)

SOIL-CEMENT SUBBASES

Considerable use is not being made of granular soil-cement subbases to prevent pumping and to increase the bearing value of the pavement foundation. Cement was added to materials 1, 2, 3, and 4 to determine to what extent the cement would improve their general performance and reduce densification. The soil-cement subbases, 6 in. thick, were placed at AASHO standard density and optimum moisture. A summary of the test data after 450,000 load repetitions is shown in Table 9. In this table the term "composite densification" is again used to indicate a lowering of the elevation of the top of the subbase. The very small changes in the elevation of the soil-cement are attributed to permanent deformation in the subgrade, rather than to densification of the soil cement.

TABLE 9

DENSIFICATION OF SOIL-CEMENT SUBBASES

Type Material	Cement by Wt (%)	Composite Densification (in.) With Cement	Without Cement
1 Concrete sand + 6% silt	4.0	0.005	0.14
2 Dense-graded gravel	5.3	0.01	0.22
3 Open-graded sand and gravel	4.0	0.005	0.13
4 Dense-graded limestone	5.3	0.04	0.17

After 500,000 load applications for each soil-cement material the concrete slab was lifted from the subbase to destroy any bond that might exist between the two materials. In each case, there were areas where the cement subbase adhered to the slab. The unbonded slab was then replaced on the subbase, a continuous flow of water was fed onto the subbase, and the loading was continued. As many as 500,000 additional load applications (making a total of 1,000,000 repetitions) were applied without an appreciable increase in the magnitude of densification and without the occurrence of pumping. It was of particular significance that the addition of cement to materials 2 and 4 prevented the pumping exhibited by these materials in the subbase densification tests.

SUBGRADE SOIL PRESSURE

In addition to the 12-in. diameter plate bearing tests made on the subbases before and after repetitive loading (Table 5), supplementary plate tests were made on samples of the subbases compacted to thicknesses ranging from 2 in. to 15 in. These tests

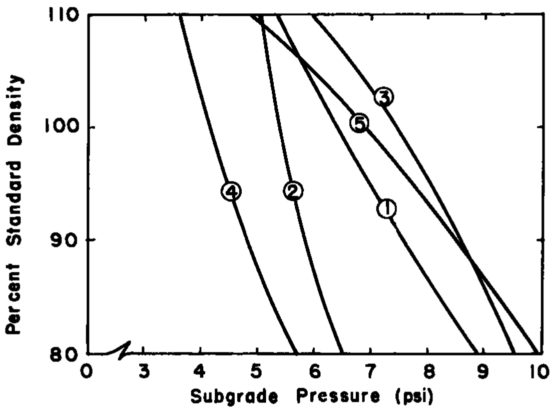


Figure 22. Influence of subbase density on pressure transmitted through various types of subbase materials. (6-in. subbase layer - 10 psi on 12-in. dia. plate.)

were conducted with the 12-in. plate centered over the subgrade pressure cell. This procedure permitted the development of data on the transmission of pressure through various subbase materials to the subgrade. The data shown in Figures 21 and 22 were obtained by applying 10 psi to the plate on the subbase, and reading the pressure indicated by the subgrade pressure cell. The data indicate that the magnitude of subgrade pressure is related to the cohesion of the subbase material (Fig. 22 and Table 3). Materials 2 and 4, with cohesive strengths of 4.0 to 5.2 psi, transmitted only about two-thirds of the pressure transmitted by materials 1, 3 and 5, with cohesive strengths of 1.0 psi or less.

Figure 22 shows that a decrease in the density of the subbase increased the pressure transmitted to the subgrade. This increase in subgrade pressure varied from about 23 to 50 percent. The larger increase occurred with material 1, 3 and 5 which had low cohesion. Materials 2 and 4, which had some cohesion, were able to maintain more integrity at lower densities and thus showed the smaller increases in subgrade pressure.

The subgrade pressures created by the repetitive loading of the slab exhibited the same trends of pressure transmission as determined from the plate loading tests. The magnitude of subgrade pressure varied from 5 to 10 psi for the 6-in. granular subbases, to about 0.5 psi for the granular soil-cement subbases of the same thickness, showing the improved load-distributing characteristics developed by the addition of cement.

SUMMARY

This program was conducted to investigate the influence of placement density and moisture content on the performance of various types of subbases under repetitive loading. The three factors which primarily determine the severity of the test conditions and thus the performance of the subbases are: the number and frequency of load applications, magnitude of load, and degree of subbase saturation. In these tests an adverse combination of these three factors was used to accelerate the program. The test conditions of frequent applications of a heavy load to a saturated subbase were more severe than existing or probable future field service conditions. It is well known that with less severe service conditions, satisfactory subbase performance is obtained with materials which contain more minus 200 fraction than the limiting 10 percent indicated by these tests. Therefore, the data and the conclusions do not necessarily justify changing subbase designs which have proved satisfactory under extremely severe conditions.

Subject to modification in accordance with the above comments, the following conclusions appear warranted.

Placement Conditions

A positive relationship was established between the subbase placement conditions and the magnitude of subbase densification under repeated loading which showed that an increase in placement density decreased the magnitude of densification. The average densification of the subbase materials compacted at 90, 100 and 110 percent of AASHO density was 0.232 in., 0.129 in., and 0.045 in., respectively. Thus, the densification was decreased about 80 percent by increasing the placement density from 90 to 110 percent of AASHO density. Furthermore, the larger portion of this decrease in densification occurred as a result of increasing the density from 90 to 100 percent. These data indicate the necessity of specifying a compaction requirement of at least 100 percent of AASHO density if densification under traffic is to be small. It is significant that the test densities of 100 and 110 percent of AASHO were obtained with a sled-type vibratory compactor.

The materials generally showed less densification and better performance when they were placed at optimum moisture than when they were placed at 2 percent on the dry side or at 2 percent on the wet side of optimum. Furthermore, the performance of the materials was better in all cases when they were placed on the dry side of optimum than when they were placed on the wet side of optimum.

Type and Gradation of Subbase Material

In general, the densification of the subbases was related to the permeability of the materials. The open-graded, high-permeability subbase materials showed less densification than the dense-graded, low-permeability subbase materials. However, in the case of material 5 (open-graded limestone), intrusion of the subgrade soil into the subbase affected its performance adversely, and considerable slab settlement (plotted as composite densification in Fig. 18) occurred. A 1-in. thick filter course of dense-graded limestone completely prevented the intrusion, and this combination showed the best performance of any of the materials.

The use of any of the subbase materials tested prevented pumping of the clay subgrade, although in time subgrade soil undoubtedly would have been pumped through the material 5 without a filter course. Even a 1-in. thickness of dense-graded limestone or of concrete sand with 6 percent silty soil prevented subgrade pumping. There was no pumping of the subbase, when the subbase material contained less than about 10 percent material passing a 200 mesh sieve. Subbase pumping occurred late in the saturation stage with two dense-graded, coarse granular subbases; one was a silty sand and gravel which contained 12 percent minus 200, and the other was a crushed limestone which contained 17 percent minus 200. Some indication of pumping was observed with a subbase of sand containing 12 percent minus 200. The performance of the three other predominantly sand subbases was very good.

The addition of cement to four granular subbase materials reduced the densification of these materials to an insignificant amount and eliminated pumping from two of them which had shown pumping without the added cement.

Pressures transmitted through the 6-in. thick subbases to the subgrade were from 5 to 10 psi for the granular materials and 0.5 psi for the soil-cement subbases.

Principal Findings

1. Compaction of granular subbases to at least 100 percent of AASHO standard density was found necessary to assure that only minor densification would occur under repeated loading.
2. Total densification increased as the thickness of subbase increased. Therefore, particular attention must be directed to compaction of thick subbases to preclude serious densification under traffic.
3. Open-graded, high-permeability subbases showed less densification than dense-graded, low-permeability subbases.
4. Subgrade intrusion into coarse-graded subbases was prevented by a 1-in. filter course of dense-graded material.
5. Under severe conditions of test, some dense-graded subbases pumped. Pumping did not occur when the subbases contained less than about 10 percent passing a 200-mesh sieve.
6. Granular soil-cement subbases did not pump; they showed little or no densification, and, as compared to granular subbases, they reduced greatly the pressure transmitted to the subgrade.

ACKNOWLEDGMENTS

The initial planning of this project was undertaken by L. D. Childs. Recognition is also given to Ray Boldt and R. D. Ward for their careful attention to conducting the tests. The entire program was under the direction of E. F. Felt, Manager, Transportation Development Section, and Douglas McHenry, Director of Development.

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Appendix

DETERMINATION OF MAXIMUM AND MINIMUM DENSITY

Maximum density was determined by a vibration method. A 5-lb saturated sample was placed in a 1/10-cu ft cylindrical container. The bottom of the container, which was perforated, was covered by a 40-mesh screen and a filter cloth which permitted drainage of the specimen without loss of soil fines. A surcharge weight of 3 psi was placed on the specimen and the container was bolted to a wooden platform. The assembly was vibrated 30 minutes at a frequency of 60 cycles per second and an amplitude of about 0.1 in. The final volume and moisture content were determined, and the unit weight was computed.

Minimum density was determined by a gravity method. A funnel was filled with 12 lb of air-dry soil and discharge into a 1/10-cu ft cylindrical container until it overflowed. The funnel, having a 2-in. by 2-in. tube initially resting on the bottom of the container, was raised spirally permitting the soil to fall in place approximately 1 in. below the point of discharge. Excess material was removed by screeding without jarring the container or compressing its contents. Unit weight was computed from the amount of soil contained in the mold.

Effect of Base Course Gradation on Results of Laboratory Pumping Tests

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E. J. YODER, Purdue University

This paper reports the results of a laboratory study to investigate the performance of a variety of base course samples with different gradations when these samples are placed over a standard subgrade soil and subjected to repetitions of load in such a way as to induce the pumping of fine soil to the base course surface and the intrusion of subgrade soil into the interstices between base course particles.

A good correlation existed between large deflections of a subgrade-base course system and either base course pumping or subgrade intrusion. Specimens with very open-graded gravel bases were subject ot intrusion of subgrade soil, and specimens with dense-graded gravel bases (excess of 3 percent by weight finer than the No. 200 mesh sieve) demonstrated pumping of fine soil sizes to the base course surface. An optimum gradational range exhibited neither base course pumping nor significant subgrade intrusion. Test specimens with coarse sand base course samples performed satisfactorily over a wider range of gradation than those with larger sized gravel bases. Test results were compared with existing filter criteria for thin base courses.

●THE DESIGN of granular base courses for use under rigid highway pavements, from the standpoint of gradation, has for the most part developed from the results of numerous pavement performance surveys and several highway test sections (1, 18, 19, 29). These performance surveys have been conducted and these test sections built by the highway departments of the various states and by public and private research organizations. The results of such endeavors have been general in nature, in most cases indicating trends, and have been incorporated into the specifications for base course materials of various agencies, most often in the following forms: (a) limits on the allowable percentage of different size fractions, (b) restrictions on the quantity of material finer than the No. 200 mesh sieve or finer than 0.02 mm, (c) a maximum allowable plasticity index of the finer soil sizes and (d) general direction toward the use of either open or dense-graded materials. Often, however, specifications have necessarily been tempered by the quality of locally available materials, the economics of altering the gradation of existing natural materials and expedients of construction. The concern of engineers regarding the gradation of granular base courses has developed primarily through attempts to control the detrimental effects that often accompany frost penetration and pavement pumping. As a result, gradational requirements vary from area to area with the factors that influence these problems (differences in soil conditions, rainfall, temperature, traffic, and available materials of construction) and with the experience of local engineers.

In September 1953, the Purdue Research Foundation entered into a contract with the Arctic Construction and Frost Effects Laboratory of the Corps of Engineers, U.S. Army, to study base course requirements for rigid pavements constructed over frost-susceptible subgrades. The purpose of this study was to provide data either to substantiate or to revise existing criteria relative to required thickness and quality in areas which experience significant frost penetration.

[1] Formerly Research Assistant, Joint Highway Research Project, Purdue University.

This investigation consisted of two phases, an extensive field and office correlative study of highway and airfield pavements (32) and a statistical laboratory study to evaluate the relative influence of various subgrade and base course factors on the pumping of rigid pavements (17).

The former included a literature search of current airfield and highway practices, field observations at selected air installations, and an edge sampling program and performance survey of rigid pavements in Indiana constructed with granular subgrade treatment. Data obtained from this study indicated that performance of rigid highway pavements built with granular bases is greatly influenced by the gradation of the base course material.

The laboratory study showed that the performance of a subgrade-base course system cannot be explained solely by a consideration of the direct effects of one selected factor as base course gradation, but is dependent to some extent on the interaction between a number of factors; such as, subgrade type and compaction, base course type and compaction, and the magnitude of the applied load. It was concluded, however, that if selection of a base course type is feasible in a given situation, one with an open textured gradation should be chosen since it would be apt to deflect less under repeated loads than a dense-textured base until the total deflection of the latter had increased to a point where structural failure of the overlying pavement could be postulated.

DEFINITION OF THE PROBLEM

In an investigation of this nature, it is important that the event being observed be defined so that test results may be viewed in the proper perspective. No attempt is made to review the literature pertaining to pavement pumping as this has been done many times by others. The following comments, however, present a concept of pumping action which may or may not be in contrast to that of others. It is important to note that these remarks apply only to pumping of rigid pavements built on granular bases. Pumping of fine-grained soils has received attention by many investigators and will not be discussed here.

Pumping action, with respect to rigid highway or airfield pavements constructed over granular base courses, is an inclusive term relating to the movement of finer soil sizes (sand, silt, and clay), which exist beneath a pavement surface. Removal of fine soil sizes results from the rather rapid movement of water carrying with it soil particles either in suspension or in motion as a result of hydrodynamic forces, the water moving under a pressure differential caused directly or indirectly by the deflection of a loaded pavement slab. This action may be minor and produce no significant pavement distress or it may result in severe pavement damage. This action may proceed for varying periods of time before surface evidences of it are discernible. The action is generally intermittent or cyclic in nature over a given period of time.

Pumping action may consist of one or all of the following:

1. The movement of particles from and about the base course surface. This aspect of pumping is erosional in nature and results from the lateral movement of free water along the base course surface as a result of displacement by a deflecting slab causing water and soil, upon occasions, to be ejected at pavement joints, cracks, and edges. The conditions necessary for this type of pumping to occur are: the existence of a void space between the pavement slab and the base course surface, the presence of a small amount of free water on the base course surface, and the existence of particles small enough to be moved. This type of pumping has been termed "blowing" by some investigators.

2. The movement of the finer soil sizes within and out of the base course. This aspect of pumping may result from the release of pore water pressures or by elutriation if deflections within the base course are great enough. Conditions necessary for this type of pumping to occur are: a high degree of saturation of the base course with the presence of free water to at least the base course surface, deflection in the base course sufficient to cause the development of adequate pore water pressures,

pore sizes in the base course of sufficient size to permit the movement of fine soil particles, and soil particles within the base course small enough to be moved. This type of pumping has been induced in laboratory tests (17).

3. The movement of subgrade soil into the interstices between base course particles. It has been postulated (17) that this might reslut from the release of pore water pressure developed in a softened subgrade soil resulting from the transmission of load through a granular base course to the subgrade surface. A distinction is made here between this aspect of pumping and the intermixing of subgrade and base course that may result from a kneading action. The latter is not considered to constitute pumping in terms of the definition given above but, rather, is related to local shearing failure of the subgrade soil at its surface of contact with the base course. The conditions considered necessary for this type of pumping to occur are: a high degree of saturation of the subgrade soil so that appreciable pore water pressures will result from small deflections of a loaded pavement, cohesive bonds in the subgrade soil which are not strong enough to resist the hydrodynamic forces of escaping pore water, a compressible subgrade soil, a base course which is many times more permeable than the subgrade, a source of available ground water, and pores in the adjacent base course material of sufficient size to permit the entrance and movement of subgrade soil particles.

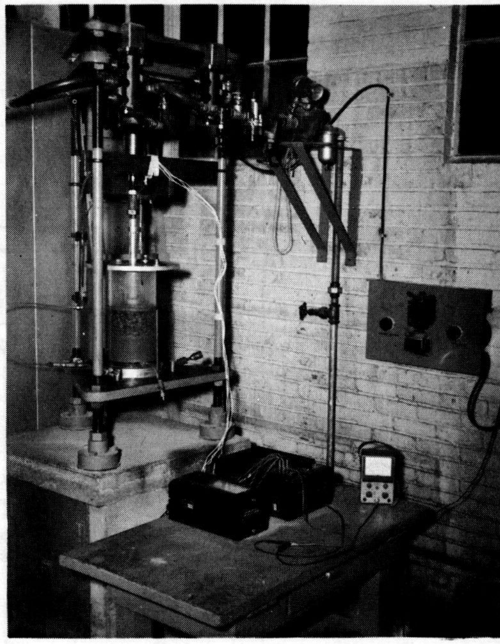

Figure 1. Repetitive loading equipment.

PURPOSE AND SCOPE

This research program investigated by means of repeated load tests, the performance of a variety of base course gradations with respect to the movement of fine soil to the base course surface and the movement of subgrade soil into the interstices between base course particles. Eight gravel base course samples, varying from extremes of open-graded to dense-graded; 7 coarse sand base course samples, similarly graded; and a series of uniform-graded samples were placed, each at a relative density of approximately 90 percent, over a standard silty-clay subgrade compacted to 90 percent of modified AASHO density, and subjected to repetitions of a 25-psi load applied at the base course surface. In each case, the load was applied through a loading piston which at all times remained in contact with the base course surface. It was possible with this type of test to induce the movement of fines through and out of the base course as well as the intrusion of subgrade soil into the interstices between base course particles by a pumping action or by mechanical manipulation or kneading at the subgrade-base course surface of contact. It was recognized that there are other aspects of base course performance for instance, permeability, which are dependent upon gradation; but these were not considered.

TEST EQUIPMENT

The loading device used for testing subgrade-base course combinations is shown in Figures 1 and 2. Essentially this piece of equipment consists of a piston mounted vertically in a loading frame and activated by compressed air. The pressure applied at the piston face could be controlled for magnitude by a regulator in the air line and

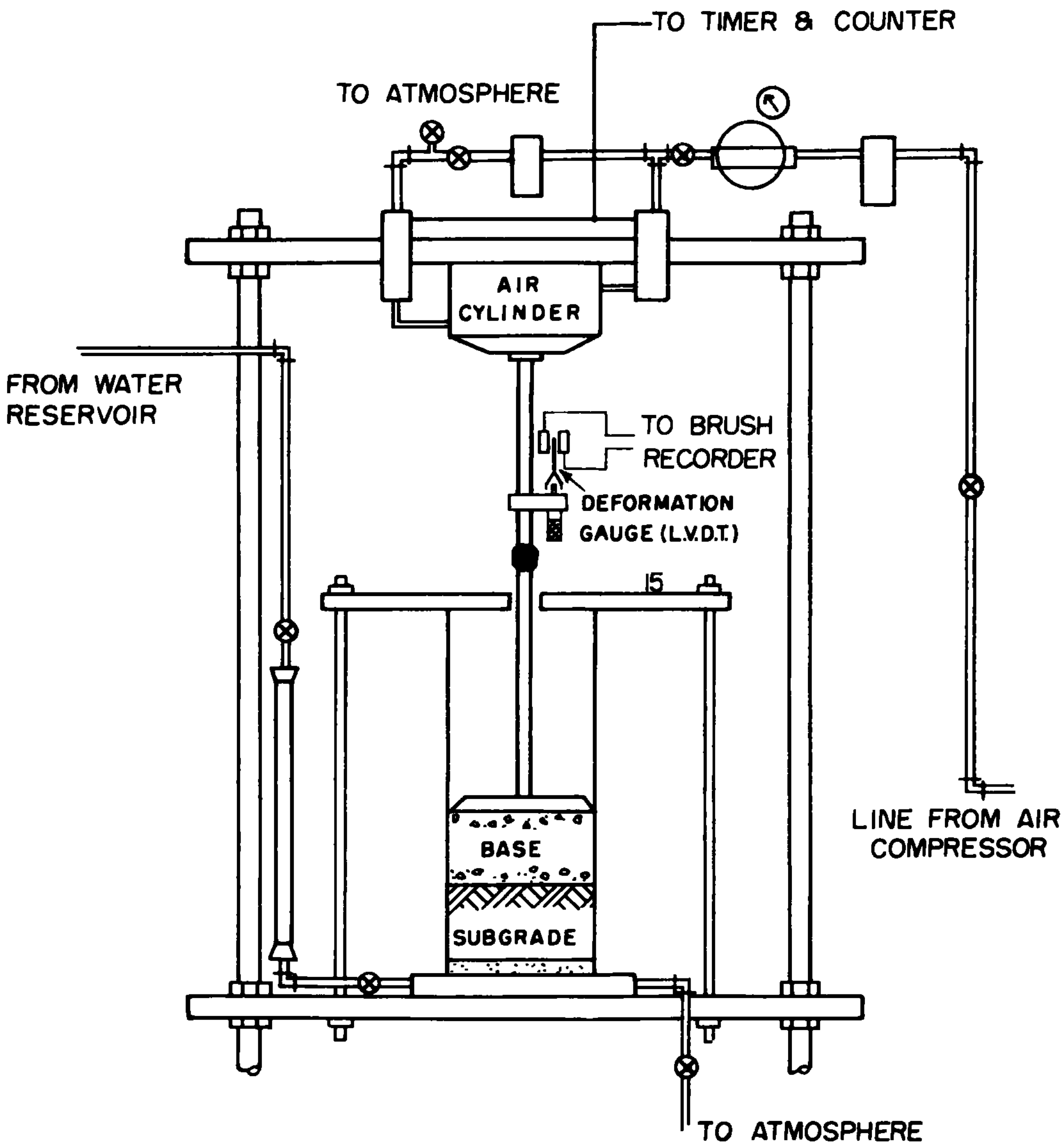

Figure 2. Schematic diagram of repetitive loading equipment.

for duration and interval by a set of inlet and exhaust valves activated electrically by a timing device (17).

Compressed air at 100- to 110-psi outlet pressure was supplied by an air compressor. From the compressor tank a 1-in. diameter galvanized pipe led through an air line filter to remove moisture and any foreign particles from the air, and thence into a pressure regulator.

From the pressure regulator the line passed through an air line lubricator and branched to enter 2 electrically-controlled valves, each of which was connected to one end of an air cylinder. This cylinder was mounted vertically to the upper platen of a loading frame and afforded a downward stroke of from 0 to 2 in. for the loading piston. A loading head attached to the loading piston applied load directly to samples of subgrade and base course placed on the lower platen beneath the air cylinder. Any desired number of loading cycles between 0 and 99,999 could be preset by means of a predetermined counter.

Adjustments were made so that only the single-acting type of movement of the loading piston was used; that is, the loading piston remained in contact with the base course surface at all times. The arrangement used to measure deflections of the subgrade base course system utilized a linear variable differential transformer with suitable recording apparatus (see Fig. 2).

MATERIALS

Subgrade Soil

The soil chosen for the molding of subgrade samples was light-brown silty clay collected in the vicinity of Lafayette, Indiana. This soil was taken from the "B" horizon of the Crosby profile which begins at a depth of about 4 in. below the ground surface and extends to a depth which may reach 50 in. The Crosby profile occupies a considerable portion of the gently undulating Tipton Till Plain of the Older Wisconsin Drift. As such, it provides the subgrade support for many miles of highway pavement in the Lafayette area. The index properties of the Crosby soil, performed on random samples of the air-dried processed soil, are as follows:

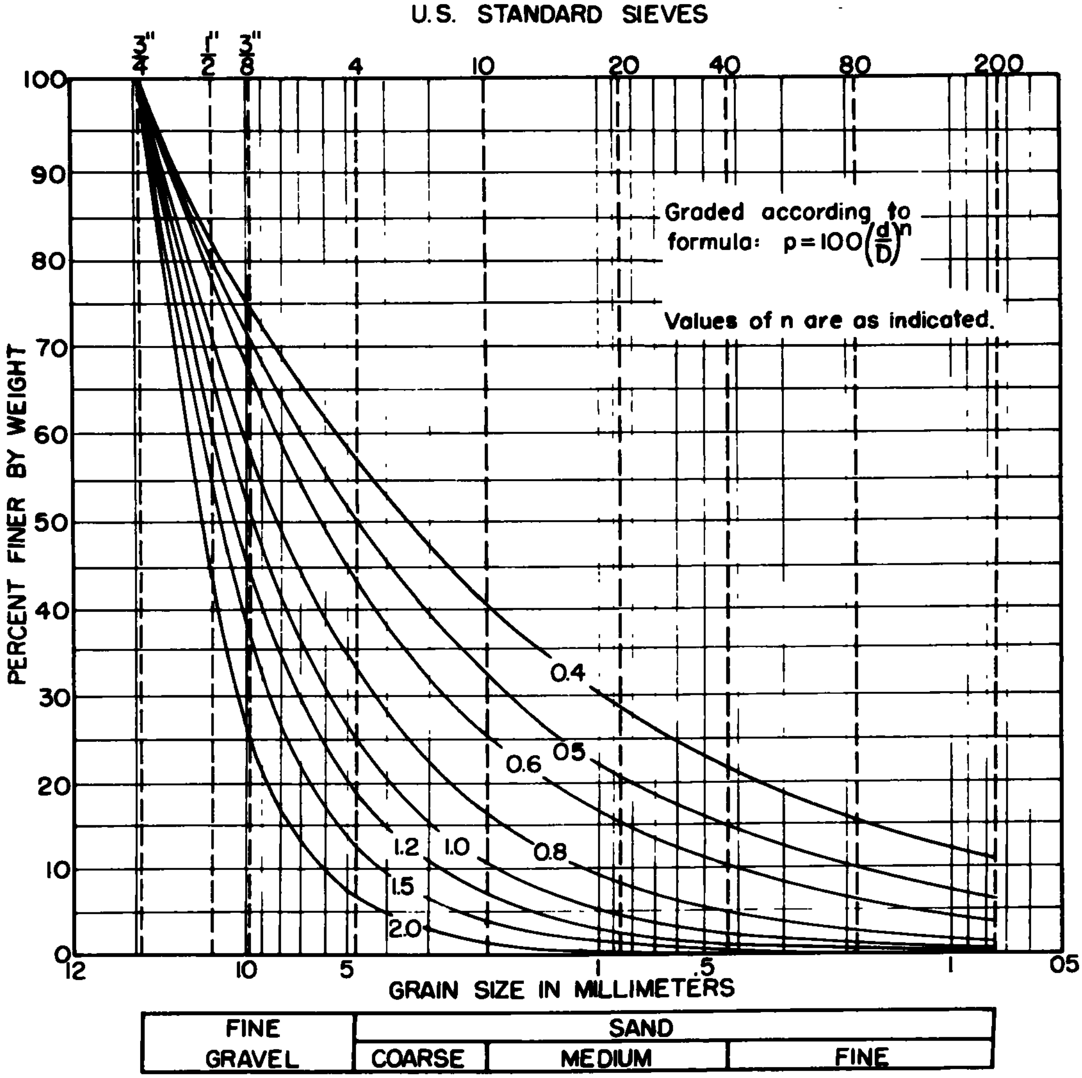

Figure 3. Base course gradations; maximum size 3/4 in.

Liquid limit =	37.2%
Plastic limit =	19.4%
Plasticity index =	17.8%
Specific gravity =	2.69
Modified AASHO maximum density =	118.8 lb/cu ft
Modified AASHO optimum moisture =	13.6%
Grain size distribution	
No. 4	100% finer
No. 200	85% finer
0.01 mm	40% finer
0.005 mm	30% finer
0.001 mm	20% finer

Base Course Materials

The material selected for use in preparing base course samples was washed glacial terrace gravel obtained commercially from a local source.

A dune sand supplied the size fraction contained between the No. 80 and No. 200 sieves and a silt of low plasticity, pedologically classed as Vigo, was used as a minus No. 200 filler. The particle shape of the individual gravel pieces ranged from subrounded to well-rounded.

For the graded materials, two maximum sizes were selected for study: ¾-in. and No. 4. A set of distribution curves was developed for each maximum aggregate size, based on Talbot and Richart's (30) mathematical expression:

$$p = 100\left(\frac{d}{D}\right)^n$$

where p is the percent of material by weight which passes a given sieve having openings of width d; D is the maximum particle size of the given aggregate; and n is a variable exponent. If n equals 1.0, the curve on a linear plot will be a straight line. If n equals 0.5, the curve will be a parabola which represents the ideal grading curve for maximum density shown by Fuller and Thompson (16).

For each of the two maximum sizes, a family of curves was obtained by varying the exponent n. The gradation curves selected for study were chosen so as to be equally spaced with respect to one another and so as to cover the region between the natural limitation of uniformity and the practical limitation of minus No. 200 material (Figs. 3 and 4).

In addition uniform base course samples were prepared from 8 different sands and gravels. Using these samples, it was possible, to a considerable extent, to eliminate the problems of segregation inherent in the use of more well graded materials. The nominal grain size of base course materials used for this series of tests ranged from ¾-in. to the No. 200 mesh sieve (Fig. 7).

PROCEDURES

Compaction Studies

The procedures adopted for determining index properties and compaction characteristics of the subgrade soil were those normally used by soils engineers.

Moisture-density relationships were determined for each of the fifteen base course gradations in accordance with the Corps of Engineers' Specification CE 807.1 (13), which requires compaction in a 6-in. diameter mold by a 10-lb hammer falling 18 in. for a total of 55 blows on each of 5 layers. Considerable difficulty was encountered in developing these compaction curves particularly for the sands where the effects of bulking make moisture content very critical in the range of moisture near the optimum content. Obvious difficulties arose in attempting to screed a level surface on the compaction cylinder where particles as large as ¾-in. were involved. Since a considerable amount of aggregate breakage occurred in developing these curves, an at-

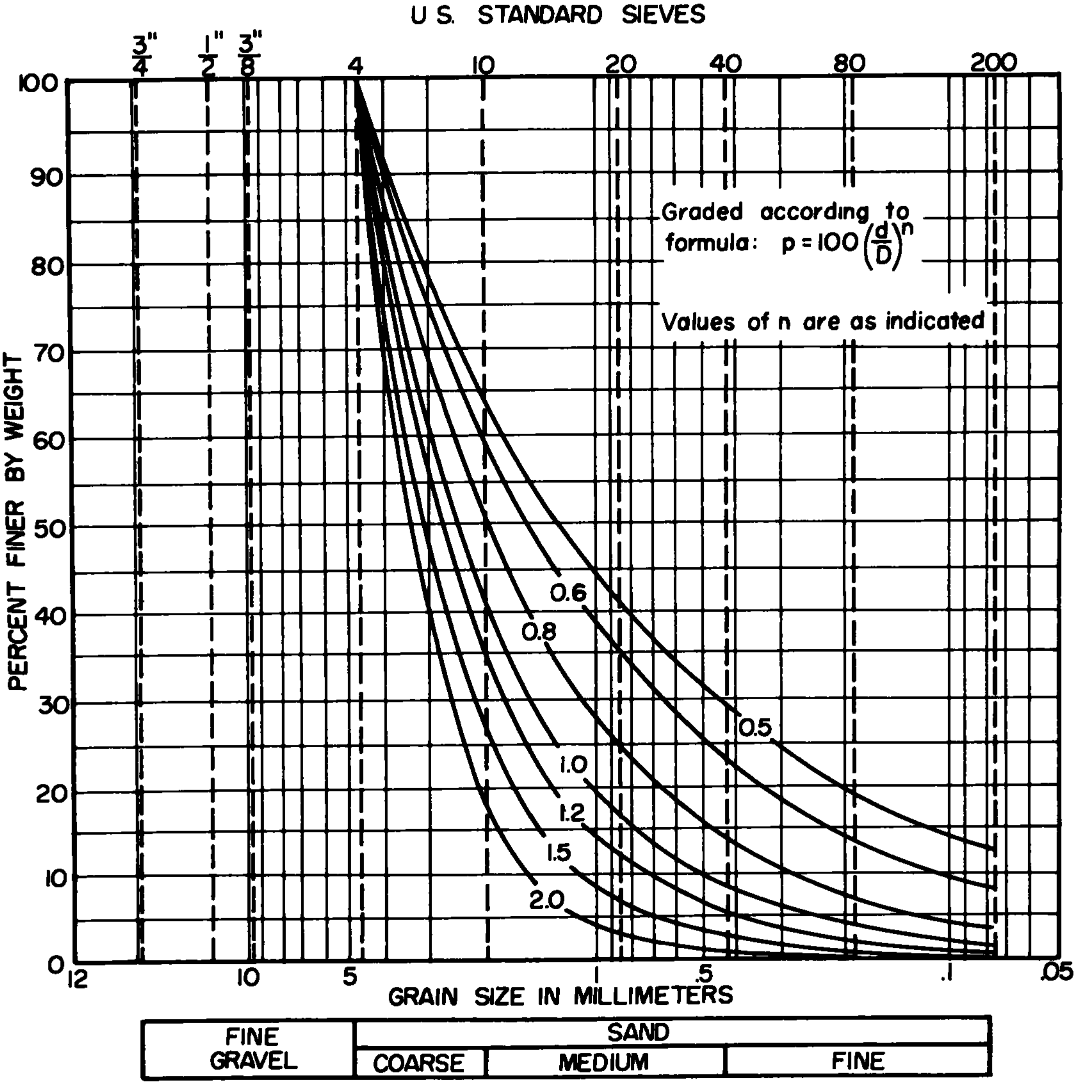

Figure 4. Base course gradations; maximum size No. 4.

tempt was also made to obtain a maximum vibrated density by vibrating a known weight of material in a lucite cylinder which was secured to the upper tray of a Gilson mechanical testing screen. The apparatus used to determine these vibrated densities is shown in Figure 5.

During the vibrating process, a surcharge weight of 80 lb was applied to the surface of the submerged sample. The sample was vibrated in increments of 10 sec, vertical measurements being taken after each 10-sec interval until a maximum density was attained.

Each base course gradation was subjected to two vibratory tests of this nature and the greater density value of the two taken as the maximum vibrated density. In nearly every instance, a maximum density was attained within 90 sec and in most cases within 60 sec. Grain size analyses of vibrated samples indicated that this short term of vibration caused a minimum of aggregate breakage even in the most open-graded samples.

Minimum densities of base course materials were determined by pouring samples loose into a cylinder of known volume, the smaller value of three trials being accepted as minimum density value.

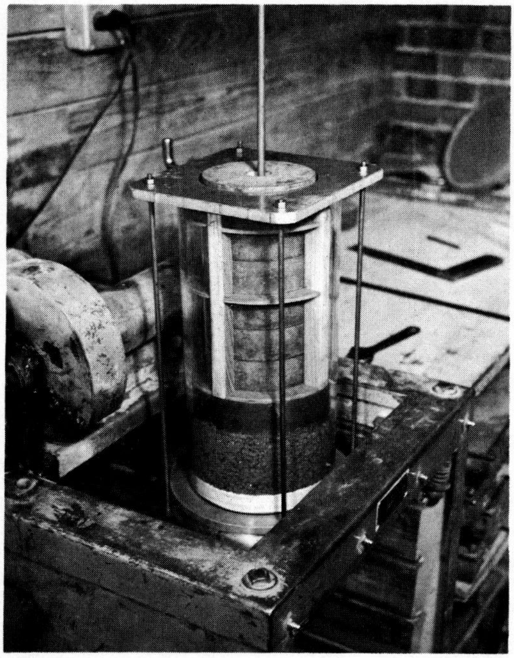

Figure 5. Vibrated density apparatus mounted in the Gibson mechanical testing screen.

Repetitive Loading Tests

All subgrade samples were compacted in a 7-in inside diameter lucite cylinder to 90 percent of the maximum density obtained by following the modified AASHO procedure. To accomplish compaction of the subgrade, the moisture content of the soil was adjusted to the optimum value, and the soil compacted statically from both ends to a height of 4 in. at the required density. Compacted subgrade samples were then permitted to absorb water for a 50-hr period prior to placing the base course.

In order to provide a significant basis for comparison of base course samples, it was decided to utilize the concept of relative density rather than compacting the base course samples to some arbitrary percentage of a dynamic test.

For simplicity, the relative density of a compacted sample was expressed in terms of density ratio, D_r, given by:

$$D_r = \frac{d_n - d_0}{d_{100} - d_0}$$

in which

d_n = dry density, pcf, of compacted specimen;
d_0 = dry density, pcf, loosest state from laboratory test for minimum density; and
d_{100} = dry density, pcf, maximum feasible density obtained in the laboratory.

This expression is analogous to that proposed by Lane (25) and is similar to Terzaghi's relative density (31, p. 27) but has the advantage that specific gravity of solids need not be determined. The two yielded results, which are quite similar, as shown by Lane (25). The concept of density ratio, as described above, was used in this study.

It was decided to compact base course samples to a density ratio of 90 percent, which might be considered a minimum requirement in order to limit objectionable consolidation and settlement under heavy traffic loads. At the completion of the subgrade soaking period, the base course material was carefully placed in two equal layers onto the compacted subgrade surface inside the lucite cylinder. Each layer was then compacted to a height of 2 in. by tamping lightly with a 2-in. diameter, 5.5-lb tamping rod; and then by applying several sharp blows with a leather mallet to a compaction head placed on the base course surface.

The combined base course-subgrade sample was next placed in the repetitive load apparatus. Before testing was commenced, the base course was saturated from above and the porous stone upon which the subgrade rested was saturated from below under a positive hydraulic head. At all times during the test, the water level in the lucite cylinder remained at or above the base course surface.

The repetitive loading apparatus was preset to deliver a 25-psi load of 0.3-sec duration to the base course surface every 4 sec. A total of 40,000 repetitions of this load was applied to each subgrade-base course combination. Periodic measurements of the permanent and elastic deformations of the system were made throughout each test by means of a linear variable differential transformer (Fig. 2).

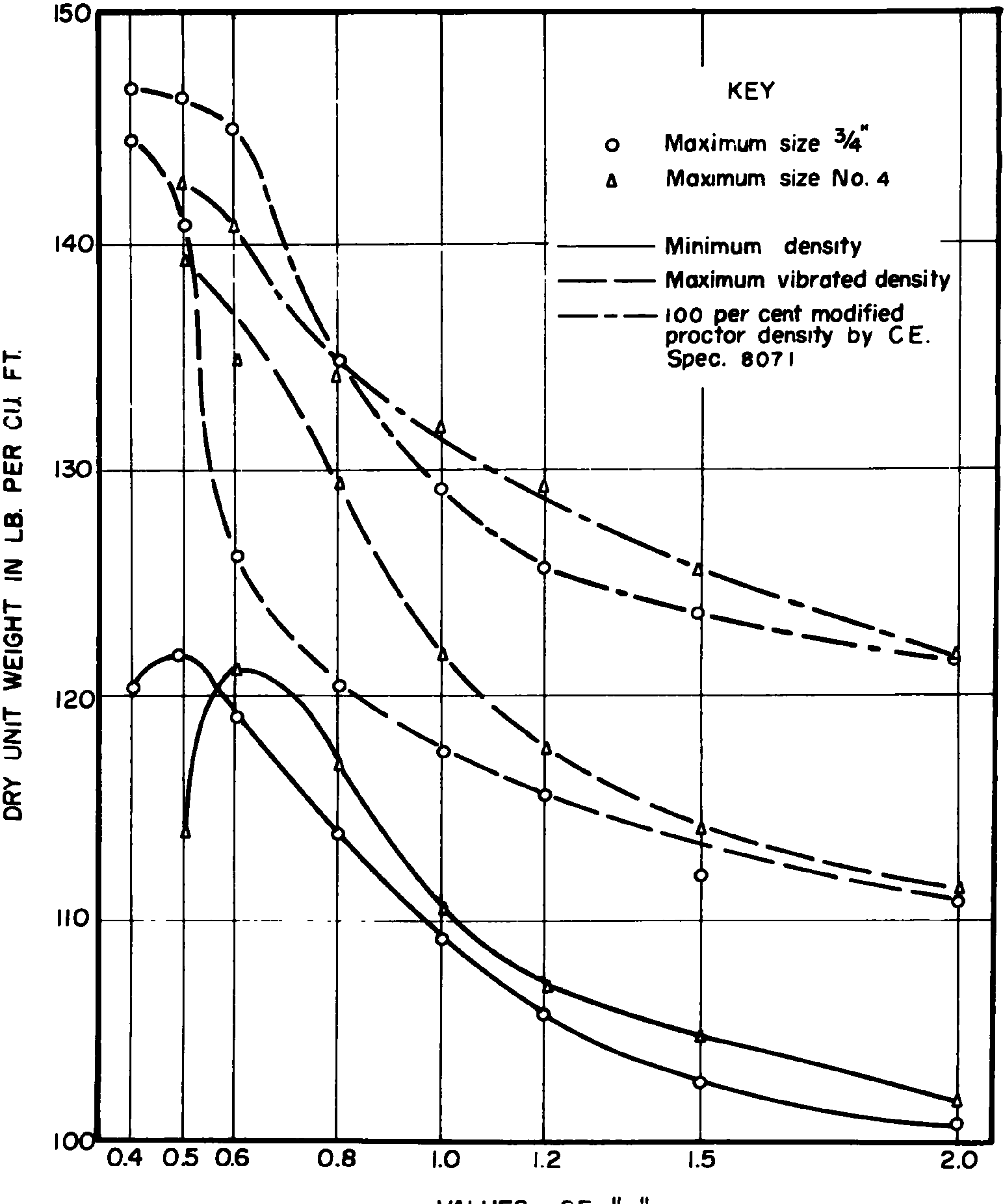

Figure 6. Comparison of density values for different base course gradations.

As each test was completed, any material that had been pumped above the surface of the base course was removed, dried, weighed, and subjected to a sieve analysis. For the more open-graded samples, note was taken of the extent of subgrade intrusion into the base course, if discernable. The entire sample was then extruded by means of a screwjack, and grain size analyses were performed on the base course to determine any changes which may have occurred in its gradation.

RESULTS

The results of the density and repetitive loading tests on the 23 subgrade-base course combinations are summarized in the following figures and tables. Figures 6 and 7 show the results of density tests performed on each base course sample. Values of maximum and minimum density are plotted against the logarithm of the n values for the graded aggregates and the logarithm of the nominal grain size for the uniform aggregates.

TABLE 1

TEST RESULTS FOR SAMPLES WITH 3/4 IN BASE COURSE

Sample	Material moved to top of base course			Weight of minus No 200 Material in base course			Increase in minus No. 200 mtrl. above subgrade (gm)
	Total weight moved (gm)	Weight minus No 200 mtrl. moved (gm)	percent minus No. 200	Before (gm)	After (gm)	Net Increase (gm)	
¾ - 0.4	206	143	69.4	676	539	-137	6
¾ - 0.5	121	103	85.1	378	290	-88	15
¾ - 0.6	81	70	86.4	204	148	-56	14
¾ - 0.8	0	0	-	64	68	4	4
¾ - 1.0	0	0	-	22	25	3	3
¾ - 1.2	0	0	-	4	39	35	35
¾ - 1.5	0	0	-	0	262	262	262
¾ - 2.0	0	0	-	0	151	151	151

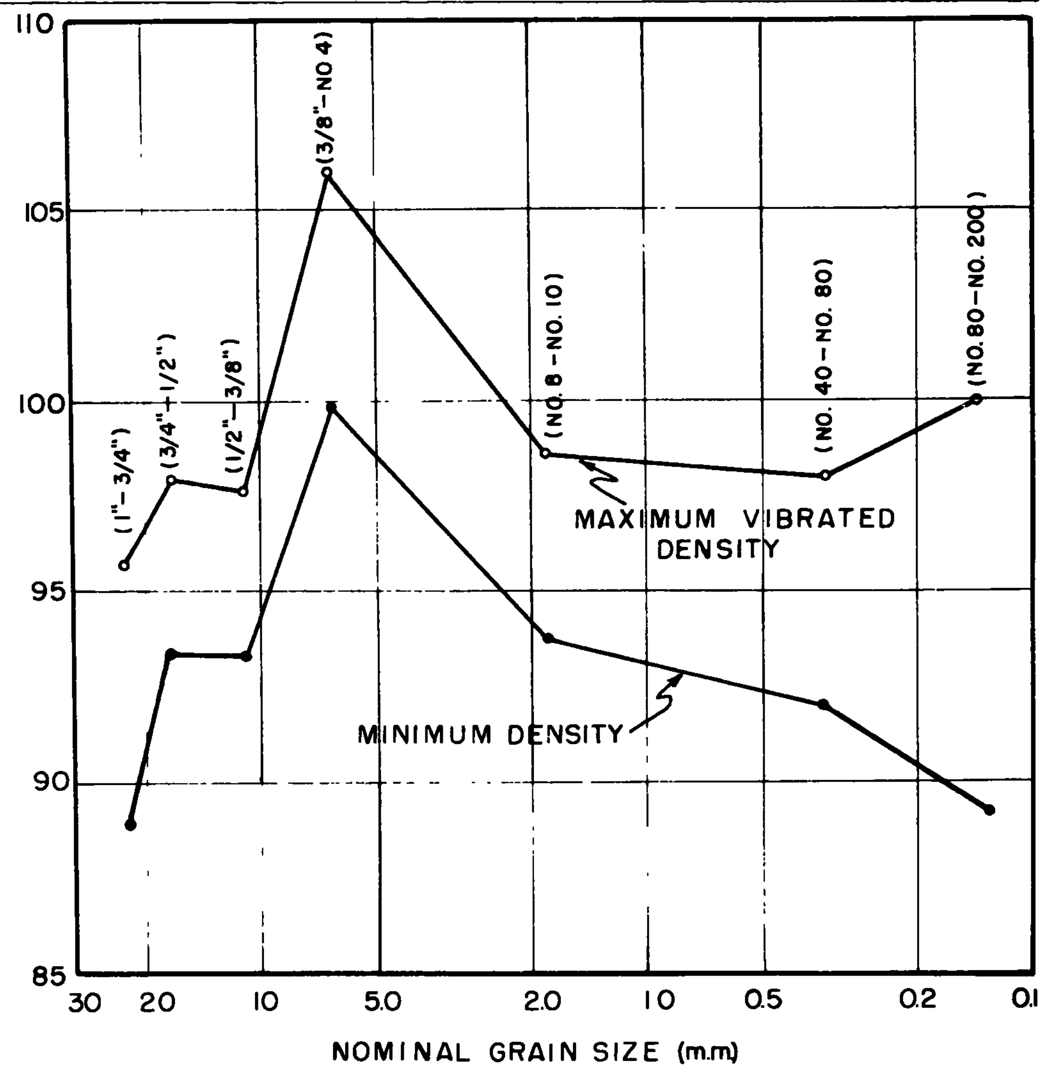

Figure 7. Density values of uniform graded materials.

In Figures 8, 9, and 10 the total deflection (permanent plus elastic) of each sample is plotted as an ordinate against the logarithm of the number of load applications. In each case, deflection readings were not commenced until after 100 applications of the 25-psi load in order to allow for adequate seating of the loading head. For this reason, the zero deflection is shown at 100 load repetitions.

At the termination of each test, grain size analyses were performed on different

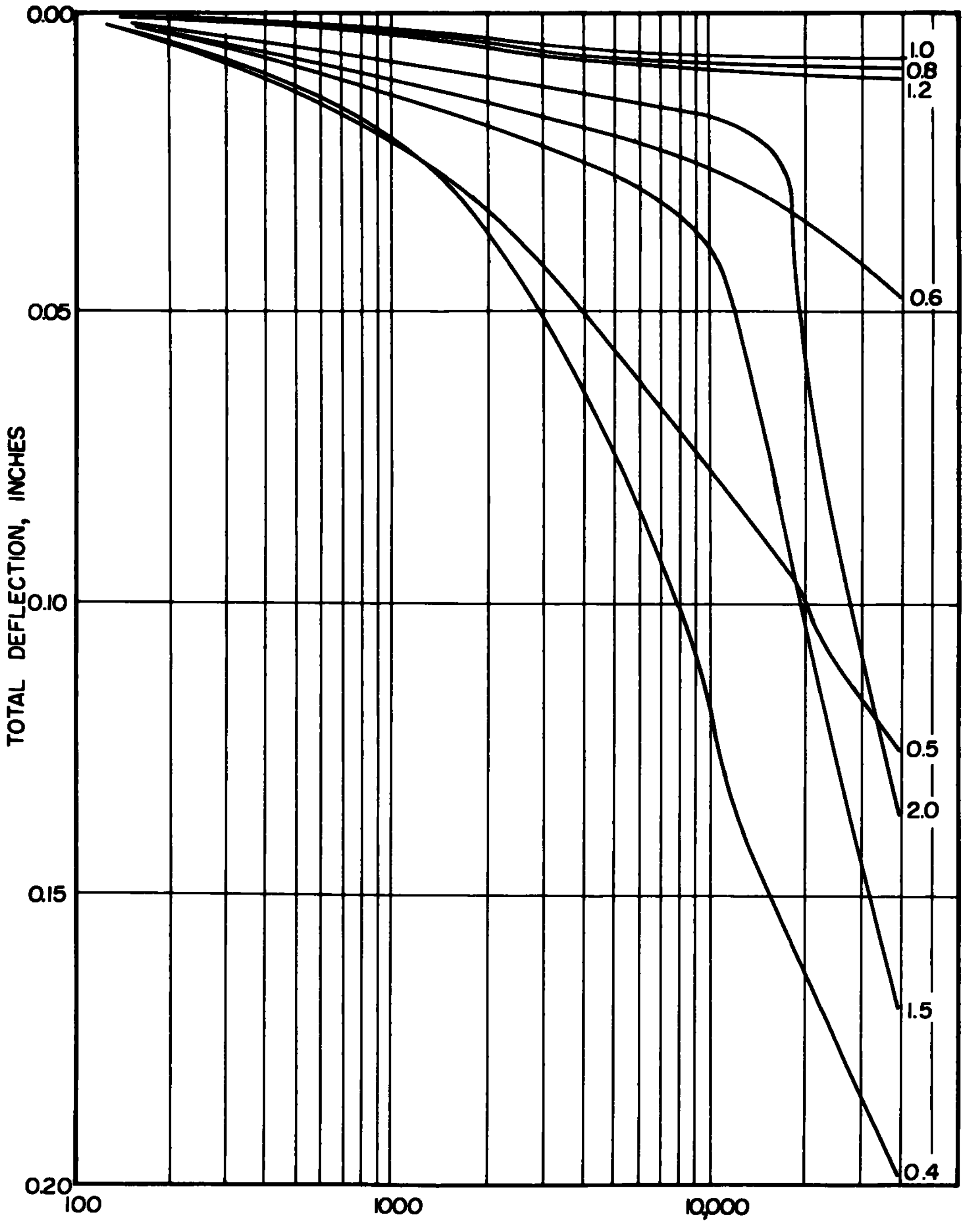

Figure 8. Deflection curves for samples with gravel base course.

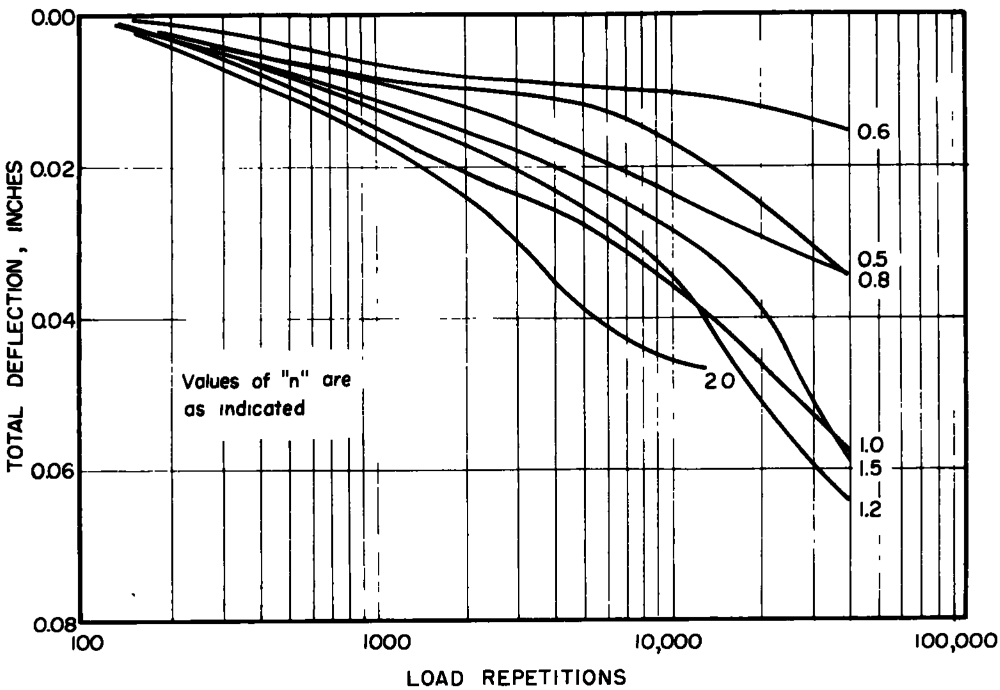

Figure 9. Deflection curves for samples with sand base course.

portions of the base course sample or, as in a few cases, on the base course as a whole to determine any change in gradation which may have occurred as a result of the repetitive loading. In addition, a grain size analysis was performed on any material which had been pumped to the top surface of the base course. From these data the increase in minus No. 200 material in the base course, the weight of minus No. 200 material moved to the top of the base course, and the increase in minus No. 200 material above the subgrade were computed and recorded in Tables 1, 2, and 3.

The transparent lucite cylinder, which was used to confine subgrade-base course

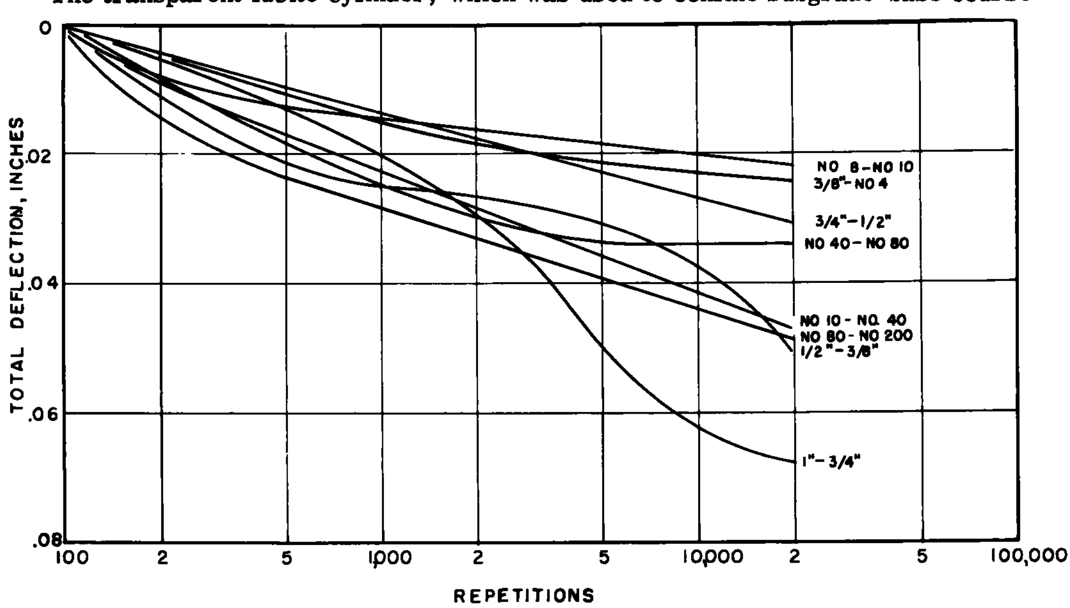

Figure 10. Deflection curves for samples with uniform graded base course.

TABLE 2

TEST RESULTS FOR SAMPLES WITH NO. 4 BASE COURSE

Sample	Material moved to top of base course: Total weight moved (gm)	Material moved to top of base course: Weight minus No. 200 mtrl. moved (gm)	Material moved to top of base course: percent minus No. 200	Weight of minus No. 200 Material in base course: Before (gm)	Weight of minus No. 200 Material in base course: After (gm)	Weight of minus No. 200 Material in base course: Net increase (gm)	Increase in minus No. 200 mtrl. above subgrade (gm)
4 - 0.5	15	8	53.4	736	723	-13	-5
4 - 0.6	0	0	-	481	478	3	3
4 - 0.8	0	0	-	200	204	4	4
4 - 1.0	0	0	-	82	80	-2	-2
4 - 1.2	0	0	-	36	51	15	15
4 - 1.5	0	0	-	9	33	17	17
4 - 2.0	0	0	-	0	24	24	24

samples, permitted a visual inspection of specimens at any time during the testing procedure. The vertical intrusion of subgrade soil into the base course was considered to be significant in five of the graded samples and three of the uniform samples. In the three most open-graded sands (n = 1.2, n = 1.5, and n = 2.0), subgrade intrusion appeared to be only slight, not exceeding ¼ - ½ in. in extent, whereas intusion to approximately 1½ in. was observed in two of the open-graded gravels (n = 1.5 and n = 2.0). These observations were supported by grain size analyses performed on the base courses after testing. The pumping of fine soil to the base course surface was observed in the case of three specimens with dense-graded gravels (n = 0.4, n = 0.5, and n = 0.6) but in only one case where a sand base course was used (n = 0.5), and then only to a small degree.

DISCUSSION

In order to evaluate properly the test results presented in the preceding pages, it is essential that the reader be familiar with the framework of test conditions in which the repeated load tests were performed, the degree of precision obtained in the preparation of samples, and the accuracy with which results were measured.

Applied Load

Each subgrade-base course sample was subjected to repetitions of a 25-psi load applied at the base course surface for a 0.3-sec duration and at a 4.0-sec interval between load applications. The 0.3-sec load duration was short enough to approximate the sinusoidal type load-time characteristics of a deflecting pavement slab which accompany the passage of vehicular traffic and was sufficiently long to allow development of the full 25-psi load before release. A 4.0-sec interval was also more than adequate to allow complete release of one load before the application of another. A 25-psi load intensity may be somewhat in excess of that generally measured under a rigid pavement, which is at least 8 in. in thickness (5, p. 45); but it was chosen in order to magnify differences in performance between different subgrade-base course samples. Such a load intensity would not, however, be unreasonable for the construction period, which time may be critical in many respects from the standpoint of over-all pavement performance and quality.

TABLE 3

TEST RESULTS FOR SAMPLES WITH UNIFORM-GRADED BASE COURSE

Sample	Increase of minus No. 200 above subgrade (grams)
1-in. - ¾-in.	-
¾-in. - ½-in.	163
½-in. - ⅜-in.	95
⅜-in. - No. 4	45
No. 8 - No. 10	21
No. 10 - No. 40	17
No. 40 - No. 80	10
No. 80 - No. 200	10

Subgrade Compaction

All subgrade samples were compacted to approximately 90 percent of the maximum dry unit weight obtained by the modified AASHO test (2). The Corps of Engineers, U.S. Army (9) requires this degree of compaction as a minimum for both cut and fill sections with the exception of those composed of cohesionless sand. This requirement, however, may be slightly below the average degree of compaction required by the various state highway departments.

Base Course Compaction

All base course samples were compacted to a density ratio of 90 percent which involved determination of the values d_0 and d_{100}. The values obtained for minimun density, d_0 can be considered as highly reliable since the procedure followed is one that is well accepted (17, 25) and in no case did the weights obtained from successive trials with the same sample vary by more than 0.06 lb. Determination of maximum density values, d_{100}, however, was admittedly quite arbitary, being in each case an average of the maximum modified AASHO dry unit weight (according to CE Spec. 807.1) and a maximum dry unit weight obtained by a procedure of mechanical vibration.

The maximum dry unit weight obtained by dynamic compaction according to CE Spec. 807.1 has been in the past taken for d_{100} (17), but in this case it was suspected to give values somewhat higher than the true maximum density, particularly for the more open-graded materials. The dynamic compaction used is a rather severe test, causing a considerable amount of aggregate breakage such that the densities obtained are not those of the original gradation but, rather, of some new gradation resulting from an alteration of the original material. Shelburne (28) showed that the breakage of aggregate due to mixing, rolling, traffic, and the Los Angeles Abrasion Test results in gradations which tend to approach Fuller's curve of maximum density. This same tendency could be presumed to exist in the dynamic compaction test. If so, it would be reasonable to assume that one would obtain unit weights in the dynamic test which would be higher than had no breakage occurred. Since the dynamic compaction test performed according to CE Spec. 807.1 applies considerable agitation and compactive effort to a granular sample, it is conceivable that unit weights approximating a maximum density could be obtained if no breakage were to occur and that densities are obtained in this test which are probably higher than for the unaltered material.

As a matter of interest, grain size analyses were performed on two samples of each gravel and sand base course gradation after having been subjected to dynamic compaction according to CE Spec. 807.1. The results of these analyses are shown in Table 4 as a tabulation of increase in percent finer than the various sieve sizes against base course gradation. It can be seen that the greater aggregate breakage does occur in the more open-graded materials with the production, in each case, of amounts of minus No. 200 material in excess of 1.2 percent and in most cases more.

Optimum moisture content has little significance for these materials from the standpoint of compaction control since any water in excess of that required to coat the aggregate surface is not retained but merely passes through the sample. These observations plus the aggregate breakage encountered and the inherent difficulties involved in developing compaction curves for this sort of material strongly support the rather well-accepted contention that such a dynamic compaction test is of little value in clarifying the true density characteristics of a fairly clean granular material.

The vibrated density test which was used, on the other hand, was believed to give values which were lower than true maximum density. It has been pointed out (17) that a greater surcharge and intensity of vibration would probably result in higher dry unit weights. One advantage of this test, however, lay in the fact that a maximum density could, in most cases, be obtained within 60 sec of vibration. Grain size analyses performed on samples which had been vibrated revealed that this short term of vibration resulted in only minor amounts of aggregate breakage.

The method of determining d_{100}, thus, was arbitrarily chosen and may be subject to some criticism. Fortunately, however, the degree of base course compaction was found to have little direct effect on the values of the measured variables in such a test (17). It was observed in this investigation that the principal effect of a lower degree of base course compaction was to cause a greater initial settlement within the first 100 or 200 loadapplications, presumedly, within the base course primarily. Thus,

TABLE 4

TABULATION OF AGGREGATE BREAKAGE IN THE DYNAMIC COMPACTION TEST (GRADED BASE COURSE MATERIALS)

Values of n	Sieve size - increase in percent finer than ¾ in.	½ in.	⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200
Maximum aggregate size, ¾ in.									
0.4	0.0	-	2.3	3.6	1.1	2.3	2.3	1.8	1.4
0.5	0.0	-	0.2	3.1	0.5	2.4	2.5	2.2	1.6
0.6	0.0	-	4.2	6.0	2.1	2.9	2.7	2.3	1.6
0.8	0.0	0.4	6.1	8.3	4.1	3.3	2.9	2.6	1.8
1.0	0.0	5.6	12.1	12.1	6.3	5.2	4.7	4.5	3.7
1.2	0.0	9.7	15.5	15.7	8.1	6.1	5.2	4.5	3.7
1.5	0.0	12.0	19.2	19.6	9.3	6.9	6.1	5.4	4.8
2.0	0.0	9.9	11.9	11.9	3.7	2.6	2.1	2.0	1.6
Maximum aggregate size, No. 4									
0.5	-	-	-	0.0	1.0	3.4	4.0	3.9	1.2
0.6	-	-	-	0.0	2.8	3.8	4.9	4.3	1.6
0.8	-	-	-	0.0	1.8	4.0	5.3	4.6	2.0
1.0	-	-	-	0.0	2.6	5.2	5.1	4.8	2.2
1.2	-	-	-	0.0	3.9	5.9	5.8	5.0	2.9
1.5	-	-	-	0.0	5.4	6.1	4.0	2.7	1.6
2.0	-	-	-	0.0	9.6	7.2	4.8	3.6	2.7

any deformation or lack of deformation which might have resulted from discrepancies between base course densities as compacted and true density ratio of 90 percent could very well have been masked by the fact that deflection readings were not plotted until 100 repetitions of the 25-psi load had been applied.

A comparison of the various density values which were determined experimentally is shown in Figures 6 and 7. It is interesting to note how critical, with respect to attainable unit weights, a small change in aggregate gradation may be within certain gradational ranges.

Measurement of Test Results

Of the values measured during and immediately after each repeated load test, the deflection of the subgrade-base course systems was considered to be the most reliable. All that was required was a linear calibration between vertical movement of the transformer core and the pen of the oscillograph recorder. This calibration was checked frequently during each test and the equipment was at no time found to require adjustment. The recorded deflections, therefore, were dependent only upon the performance of the test specimens acting under the imposed conditions. No difficulty was encountered in maintaining the magnitude, duration and frequency of loading constant from test to test or for the duration of any single test. Chance variation in the preparation of subgrade samples and the difficulties already discussed in connection with the placement of base course samples undoubtedly account for a large part of the apparent inconsistencies in the test results.

The measurement of the weight of soil pumped to the surface of the base course was also considered to be highly reliable as most of this material was contained on top of the loading head and around the sides between the loading head and the cylinder wall. Observations made during testing permitted a visual measurement of the movement of the soil. The validity of the measurement of the increase in minus No. 200 material in the base course depended to a large extent on the care that was taken in separating the base course aggregate from the subgrade soil after extrusion of the tested specimen from the lucite cylinder. This was comparatively easy in the case of specimens with sand base courses where the interface was fairly well defined. In the case of specimens with gravel base courses, however, separation was more difficult and somewhat arbitrary in a few cases, particularly where subgrade soil had been extensively intruded into the more open-graded bases.

Interpretation of Results

The transparent lucite cylinder which was used to contain the test specimens allowed visual inspection of performance while the repeated load tests were in progress and

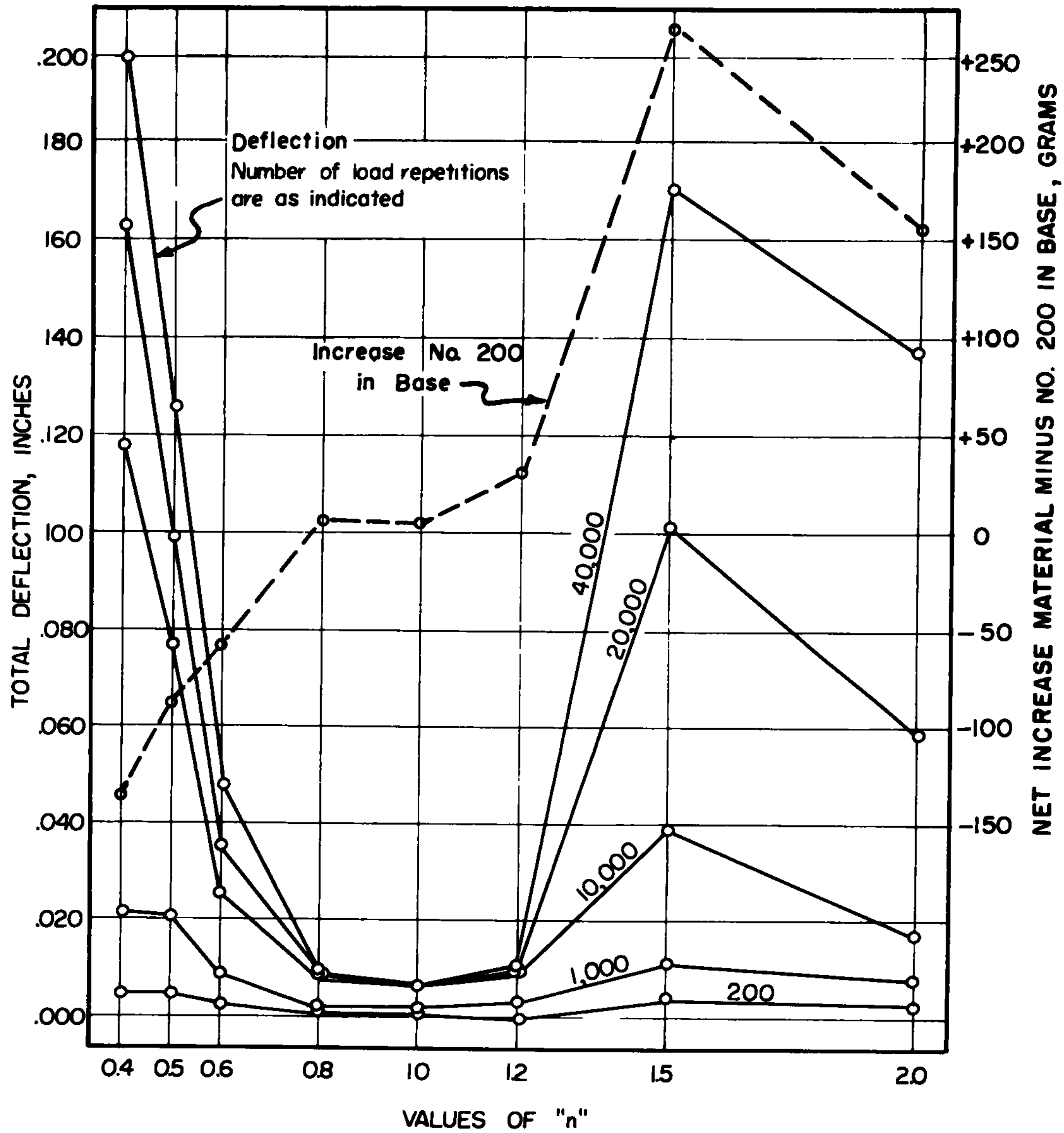

Figure 11. Deflection versus gradation for samples with gravel base course.

provided a means of observing the intrusion of subgrade soil into the base course and the movement of fine soil within the base course to the base course surface. These observations were substantiated by deflection measurements made during each test and by values measured immediately after the completion of each test.

Test specimens with gravel base courses exhibited pumping of fine soil to the surface of the three denser-graded base course samples and intrusion of subgrade soil into the three most open-graded. The uniform-graded samples exhibited subgrade intrusion in the case of the three most open gradations. Each of these two failure conditions tended to produce a characteristic type of load-deflection curve when total deflections were plotted as ordinates against the logarithm of load repetitions as abscissa. If substantial intrusion of subgrade soil occurred, the deflection curve remained, initially, rather flat and then sloped downward abruptly. This trend is seen in Figure 8 for samples n = 1.5 and n = 2.0. If, on the other hand, no intrusion occurred but substantial material was pumped to the base course surface, the curve showed deflections which were initially greater than those recorded for test specimens with the more

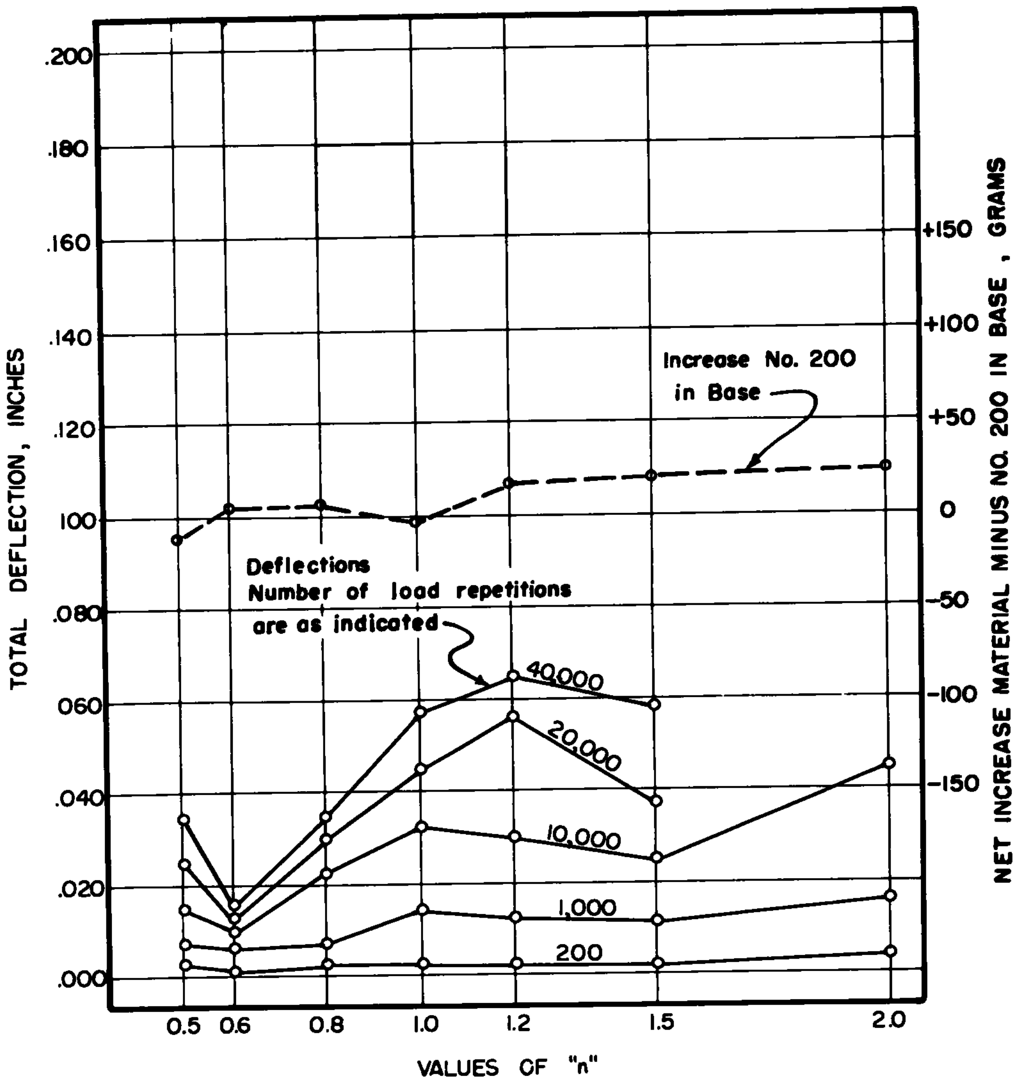

Figure 12. Deflection versus gradation for samples with sand base course.

open-graded bases in which pumping either occurred to a lesser extent or not at all (Fig. 8, samples n = 0.4 and n = 0.5). Some specimens, however, with base course samples of an intermediate gradation exhibited neither pumping nor subgrade intrusion and resulted in smaller subgrade-base course deflections. A plot of the deflections occurring in each specimen at different repetitions of load (Figs. 11, 12, and 13) indicates that there may be an optimum gradation which when compacted to a high relative density over a good subgrade will result in minimum deflections of the subgrade-base course system.

In all cases when specimens with gravel base courses were tested, a good correlation was found between large deflections and the maladies of base course pumping and subgrade intrusion, and it might be concluded that if a base course gradation could be selected that would resist these latter two effects, deflection of the subgrade-base course system could be held to a minimum in any given situation and pavement cracking and faulting resulting from lack of foundation support might be retarded.

Table 1 indicates minor increases in minus No. 200 soil above the subgrade for base course samples that experienced no visible intrusion of subgrade soil (samples

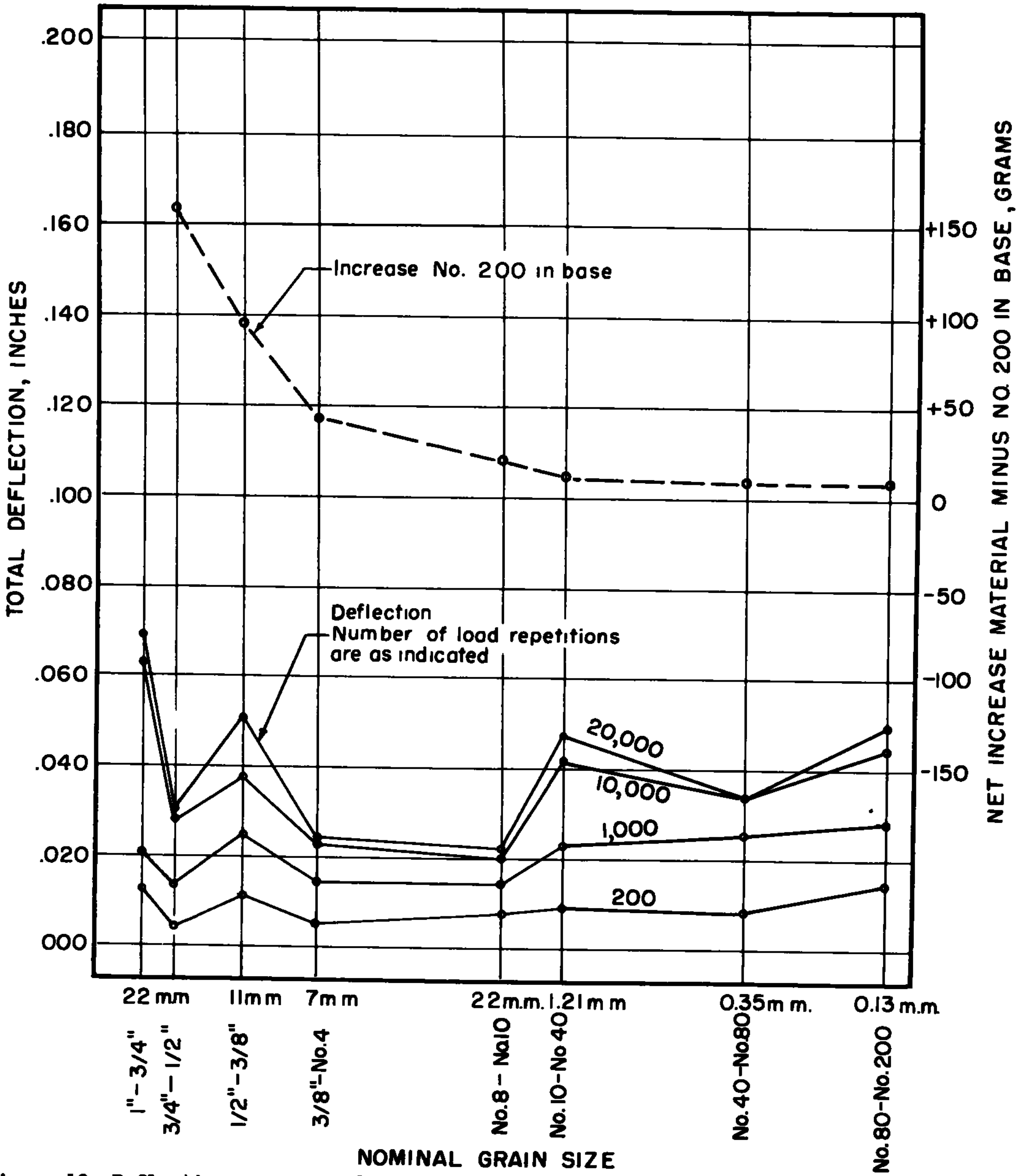

Figure 13. Deflection versus gradation for uniform base course; Crosby subgrade, 25 psi.

n = 0.4 through n = 1.0). This minus No. 200 material was believed to reflect the difficulties encountered in separating the base course sample from the subgrade soil. One rather obvious inconsistency is that sample n = 2.0 showed smaller deflections (Fig. 11) and less increase in minus No. 200 material in the base than did the less open-graded sample n = 1.5. This could probably be attributed in small part to chance differences in subgrade sample preparation but was felt to be due mainly to the great difficulty in obtaining uniformity in the base course and preventing segregation during placing which was particularly evident with the open-graded gravels.

In only one case (n = 0.5) did a test specimen with a sand base course exhibit pumping of fine soil from the base, and this was very minor in extent (Table 2). In three cases (n = 1.2, n = 1.5, and n = 2.0) a small amount of subgrade intrusion was noted. This also was shown to be minor in extent by the results of a grain size analysis performed on the base course sample after testing (Table 2). In general, specimens with sand base courses showed less material pumped to the base course surface, less intrusion of subgrade soil and small deflections over a wider range of gradation than those with gravel base courses, even though greater percentages of minus No. 200 soil existed in the sand samples than in corresponding gravels.

Intrusion of subgrade soil into the base course was experienced for the three more open samples of the uniform graded base courses. Data appearing in Figure 13 appears to substantiate the data for the graded materials in as much as the larger pore sizes reflected by the gravel samples permit a greater intrusion of subgrade soil than those of a similarly graded sand.

The one gravel sample ($n = 0.4$) and the two sands ($n = 0.5$ and $n = 0.6$), which meet the filter requirements adopted by the Corps of Engineers (10), are all dense-graded materials which appeared to allow no significant intrusion of subgrade soil during the repeated load tests. From the results of this series of tests, it appears that the criteria may be adequate for the prevention of subgrade intrusion and, in fact, somewhat conservative since it excludes most of those samples in which no significant intrusion occurred. More important, perhaps, is the fact that most materials that would meet the requirements of these criteria for the particular soil used in this study would probably be susceptible to the pumping of material to the base course surface if subjected to the conditions imposed on the specimens tested.

It can be said that a gravel base course sample graded between the limits expressed by the formulas $p = 100(\frac{d}{D})^{0.7}$ and $p = 100(\frac{d}{D})^{1.2}$ would probably perform satisfactorily (no excessive deflections due to subgrade intrusion or base course pumping) if subjected to the condition of this repeated loading test. In practice, these limits might be extended to include a wider range of gradation since less severe conditions of load would be anticipated. All else being equal, base courses constructed with a coarse sand could be expected to perform satisfactorily over a wider range of gradation than those constructed with a larger sized material.

CONCLUSIONS

Based on the results of this laboratory investigation and the experience gained in conducting it, the following conclusions appear to be justified:

1. Under the proper conditions of moisture and load, a dense-graded base course material with an appreciable percentage of material finer than the No. 200 mesh sieve could be expected to have some of its finer soil sizes removed by a pumping action and a very open-graded base course material could be expected to be contaminated by the intrusion of subgrade soil. In this series of tests, movement of soil to the base course surface occurred where an excess of 3 percent by weight of soil finer than the No. 200 mesh sieve was present in the gravel bases and where an excess of 12 percent was present in the sands.
2. Since a good correlation was found to exist between large subgrade-base course deflections and either the intrusion of subgrade soil into the base course or the removal of fine soil from the base course by pumping action, it is concluded that an appreciable degree of subgrade intrusion or base course pumping will result in objectionable deflections of the subgrade-base course system.
3. The principal effect of a decrease in the degree of base course compaction appeared to be a greater initial settlement within the early service life of the subgrade-base course system primarily within the base course.
4. An optimum range of gradation appears to exist within which a base course sample may experience neither significant subgrade intrusion nor movement of fine soil to the base course surface. For the series of laboratory tests on specimens with gravel base course, this range existed approximately between the grain size distribution curves expressed by the formulas $p = 100(\frac{d}{D})^{0.7}$ and $p = 100(\frac{d}{D})^{1.2}$. In practice, this range could be extended in anticipation of less severe conditions of loading.

5. All else being equal, base courses constructed with a coarse sand can be expected to perform satisfactorily over a wider range of gradation than those with a larger maximum particle size.

6. If a base course material is graded within this optimum range and compacted to a high relative density over a well prepared subgrade, deflections of the subgrade-base course system should not exceed that amount generally considered as tolerable for rigid pavements (about 0.05 in.).

7. With respect to the density test performed on each of the fifteen base course gradations:

a. The unit weights that can be attained with a granular material are a function of the gradation of that material with a maximum value attainable when the aggregate is graded to some approximation of Fuller's theoretical curve of maximum density.

b. There is a certain range of gradation about Fuller's curve where small changes in the grain size distribution of an aggregate markedly effect the unit weights that can be attained.

c. There is a significant amount of aggregate breakage in the dynamic compaction test according to CE Specification 807.1 (13); such that, in some cases unit weights may be indicated for a given material that exceed the maximum unit weight that can be attained for that material when its gradation is not altered by breakage.

8. It appears that the filter criteria adopted by the Corps of Engineers, U.S. Army (10) for the protection of thin base courses against the intrusion of subgrade soil are adequate and, in fact, are somewhat conservative, insofar as intrusion resulting from a kneading action is concerned.

9. The results of this investigation indicate that more consideration should be given to sand as a more satisfactory base course material than gravel.

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