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HIGHWAY RESEARCH BOARD

Bulletin 325

Compaction and Correlation Between Compaction and Classification Data

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Compaction Characteristics of Some Base and Subbase Materials

B. B. CHAMBLIN, Jr., Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville

Laboratory compaction tests using a vibrating table and field compaction experiments furnished data for a study of the compaction characteristics of base and subbase materials. Unit weights were compared to those produced by standard methods. Results indicate that laboratory vibration produces densities comparable to maximum field densities, that usual methods of correcting density for the presence of oversize particles are of limited applicability, and that density specifications should be based on the requirement that actual tests performed with apparatus yield results equivalent to those obtained in the field.

•RECENT developments in materials testing and research have indicated that some existing methods of specifying and measuring the unit weights of highway components are obsolete. For this reason, the Virginia Council of Highway Investigation and Research initiated studies of field and laboratory compaction of base, subbase, and surface course materials.

Virginia's present procedure is to require that base or subbase materials in the field be compacted to a given percentage of the standard laboratory density corrected for oversize particles. The correction formula used is given in the Appendix. It is desired to develop a laboratory method which can predict practicably attainable maximum densities.

Other States use the relative density method (1), the compaction ratio (2), or other methods.

All methods require the determination of the maximum laboratory density, and several tests have been developed for determining this density. A cooperative study by Felt (3) indicated that vibratory compaction of coarse material was most efficient. Accordingly, a vibratory compaction table, as described by Pauls and Goode (4), was constructed and used as the maximum density test apparatus by the Research Council's Soils Laboratory.

This paper concerns the compaction studies to date by the Soils Lab. These studies had four original objectives: (a) to determine the effects of water content and gradation on the density of certain granular cohesionless materials, (b) to compare the compaction characteristics of different types of aggregates, (c) to correlate the results of dynamic and vibratory compaction tests, and (d) to investigate the efficiency of several methods of correcting density for the presence of oversize particles.

For these studies, samples of 21 representative base and subbase materials were secured from Virginia's eight construction districts. The materials represent crushed and natural aggregates with a wide range of angularity, gradation, and surface texture (descriptions of these are given in Table 1). Three of the materials (60-8, 60-65, and 60-98) were from field compaction test sections.

LABORATORY TESTS

The samples of base and subbase material were compacted at five levels of gradation, four levels of water content, and with three replications, for a total of 60 tests on each material having a plus No. 4 fraction. The top size tested was ¾ in. The vibratory compaction test (4) involves vibrating an 800-g sample for at least 20 min

TABLE 1

GRADATIONS AND DESCRIPTIONS OF MATERIALS TESTED

Soil No.	Percent Passing Sieve No.							Description		
	3/8	4	10	20	40	100	200	Color	Soil Type	District
59-17	72	48	41	35	28	20	16	Pale brown	Subangular creek gravel	Salem
59-18	76	52	32	22	18	14	12	Light gray	Angular dolomite	Salem
59-19	51	37	32	27	18	9	7	Pale brown	Subangular sandstone	Bristol
59-20	59	49	33	18	13	8	7	Light gray	Angular limestone	Bristol
59-22	87	76	64	47	30	17	14	Reddish brown	Subangular gravel	Fredericksburg
59-24	64	48	37	31	25	12	7	Light gray	Angular granite	Richmond
59-25	59	37	31	26	20	6	4	Reddish yellow	Subrounded gravel	Culpeper
59-26	66	48	30	22	16	10	8	Very pale brown	Angular granite	Richmond
59-27	86	73	66	48	26	9	7	Pale yellow	Subrounded crushed stone	Richmond
59-28	80	54	33	19	15	11	10	Light gray	Angular limestone	Staunton
59-29	89	57	42	30	21	14	10	Light gray	Angular granite	Suffolk
59-30	-	-	-	100	99	21	2	Light yellowish brown	Subrounded sand	Suffolk
59-31	55	39	30	23	19	13	10	Light gray	Angular marble	Lynchburg
59-32	-	100	89	77	50	24	14	Light yellowish brown	Angular disintegrated quartz diorite	Lynchburg
59-33	80	62	50	34	20	9	4	Light yellowish brown	Subangular gravel	Richmond
60-08	100	97	94	78	46	11	2	Light yellowish brown	Subangular sandy clay	Fredericksburg
60-65	86	69	36	20	13	7	5	Light gray	Angular limestone	Culpeper
60-98	78	66	44	29	21	9	6	Light gray	Angular shalestone	Culpeper
60-117	64	46	29	20	12	9	7	Light greenish gray	Angular greenstone	Culpeper
60-120	43	36	35	34	31	14	9	Reddish brown	Subrounded gravel	Staunton
60-121	-	-	100	93	51	1	0	Light brownish gray	Subrounded sand	Suffolk

with a vertical amplitude of 0.012 in. and a frequency of 3,420 cpm with a surcharge of 1.75 psi.

Gradations included 0, 33, 67, and 100 percent plus No. 4, as well as the percent as received. Water contents ranged from comparatively dry to rather wet; water contents at the end of test are reported because water sometimes ran out during the test.

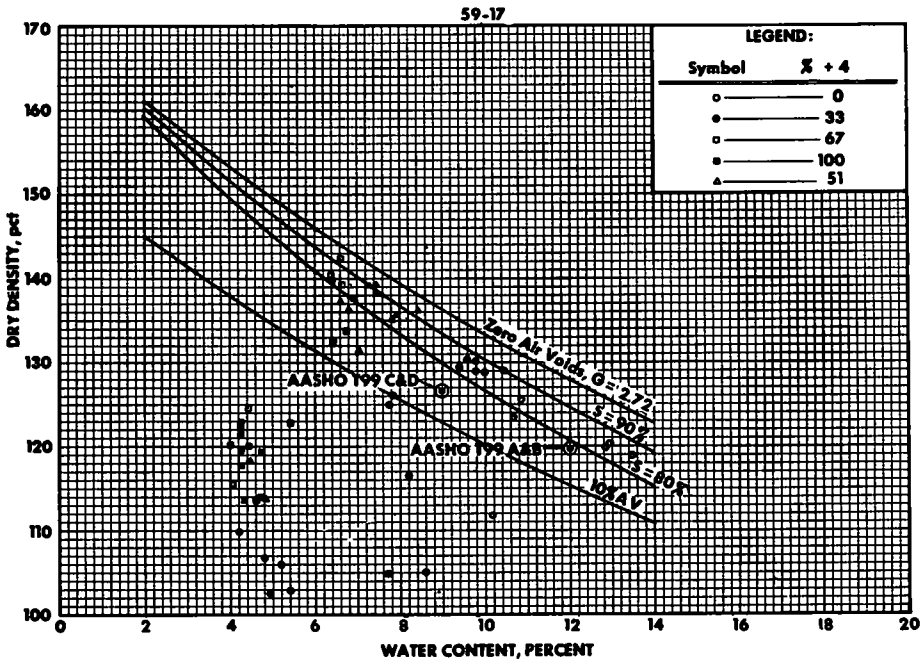

Figure 1. Laboratory test results, sample 59-17.

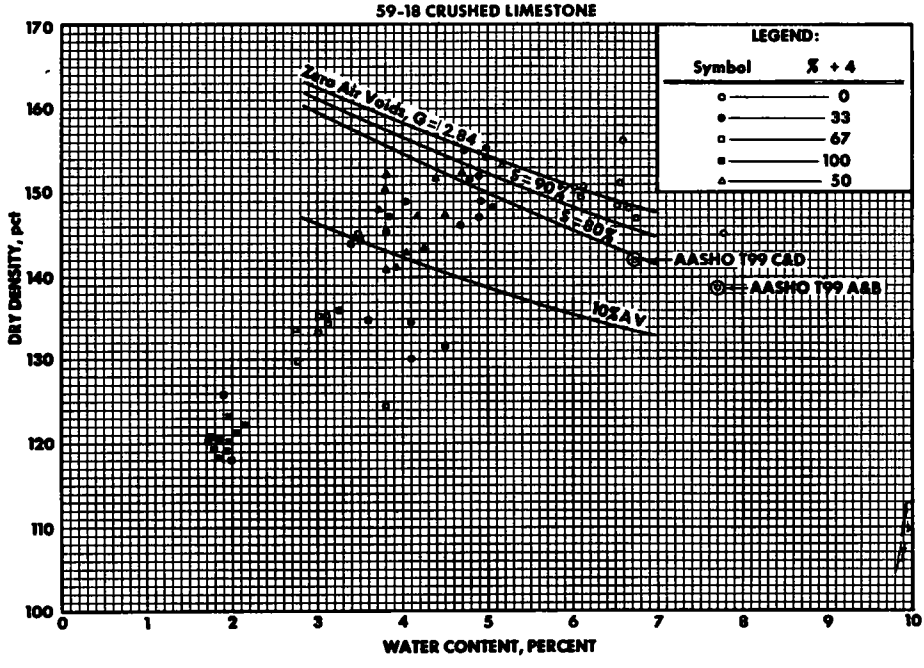

Figure 2. Laboratory test results, sample 59-18.

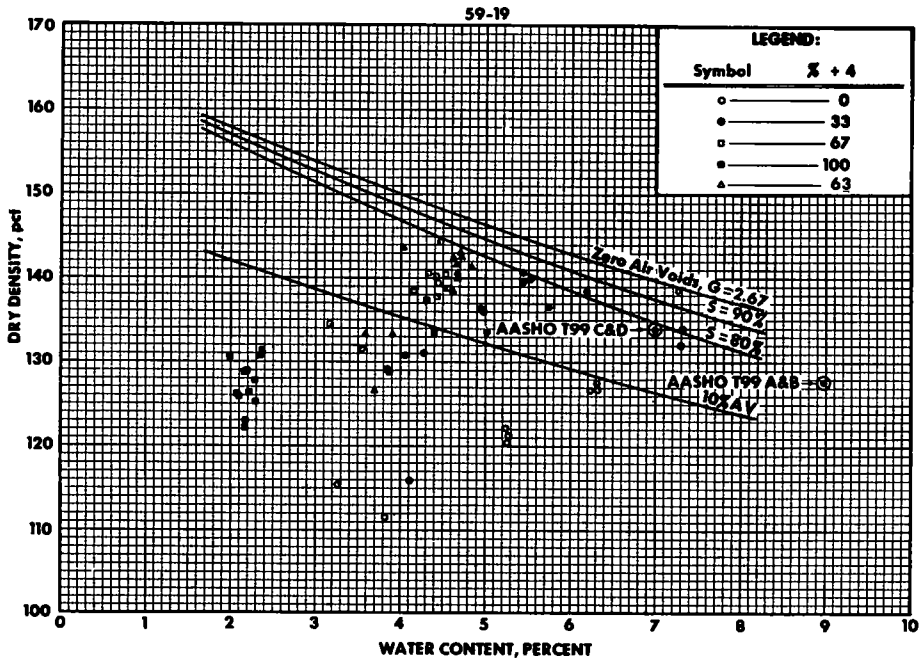

Figure 3. Laboratory test results, sample 59-19.

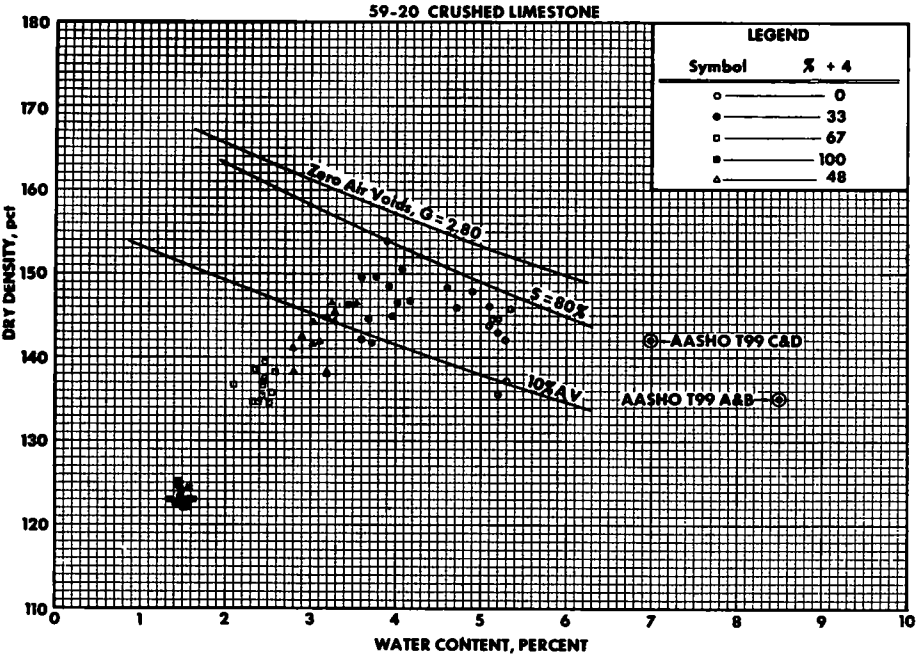

Figure 4. Laboratory test results, sample 59-20.

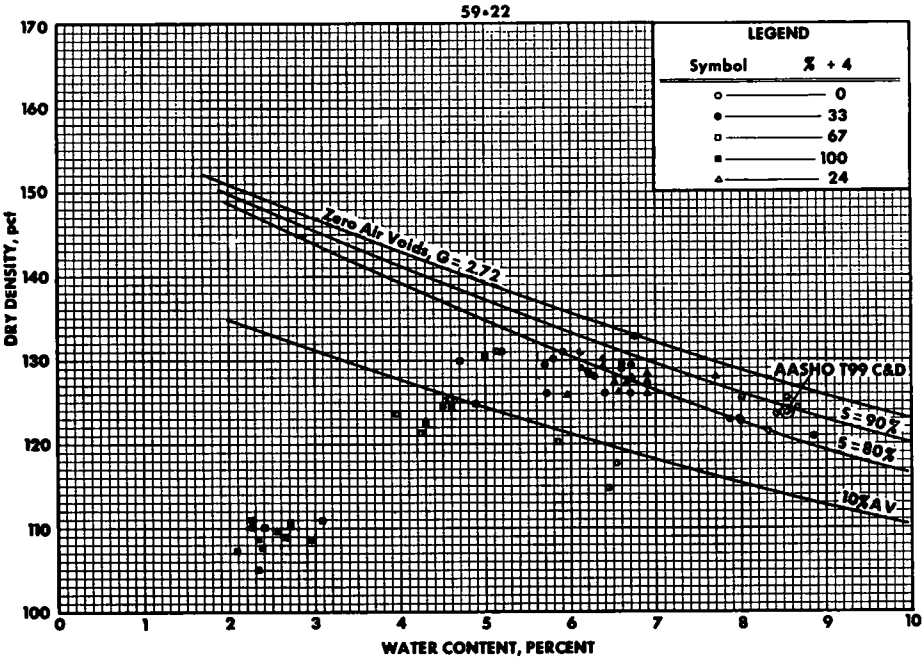

Figure 5. Laboratory test results, sample 59-22.

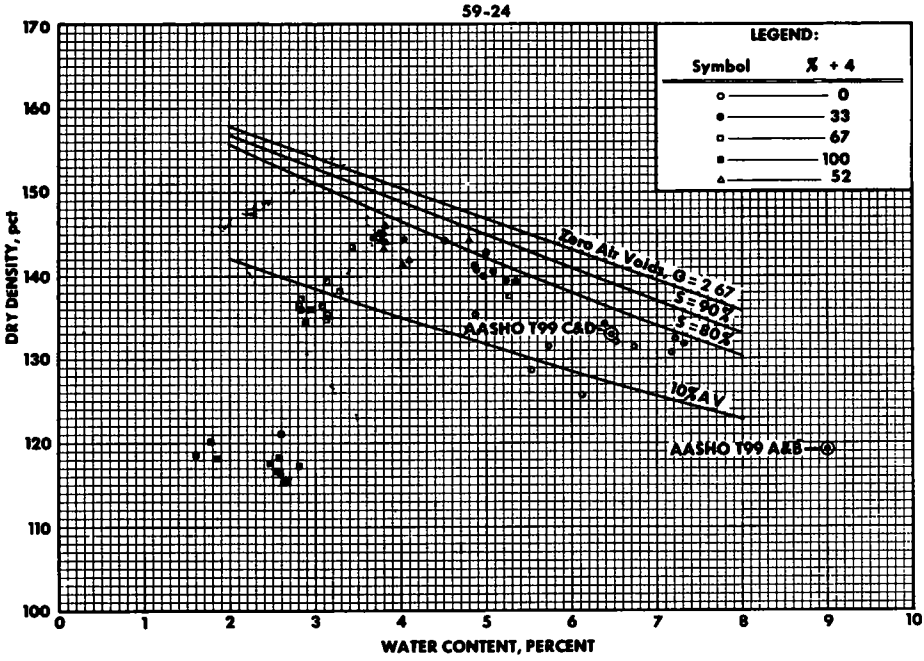

Figure 6. Laboratory test results, sample 59-24.

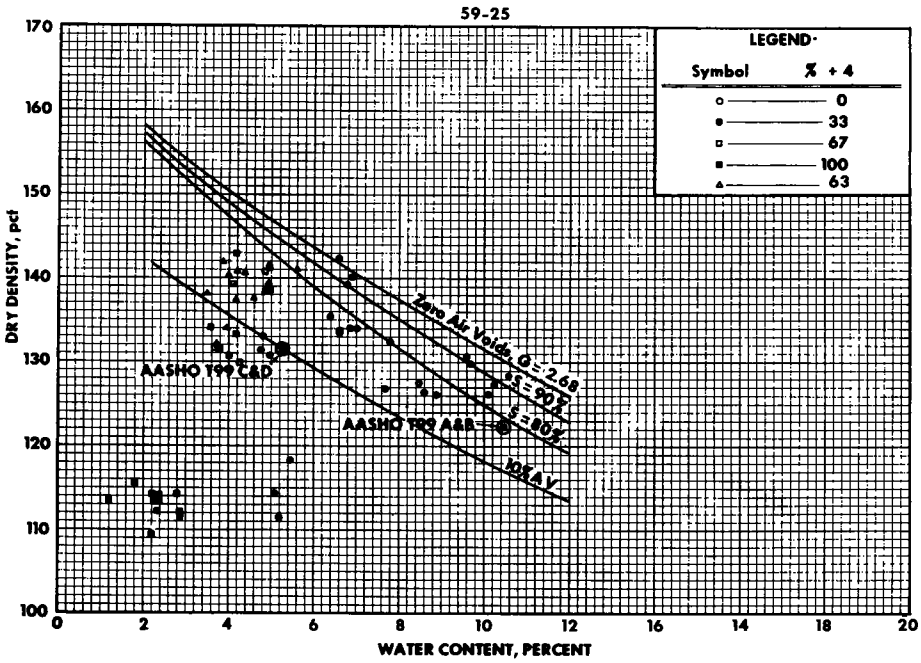

Figure 7. Laboratory test results, sample 59-25.

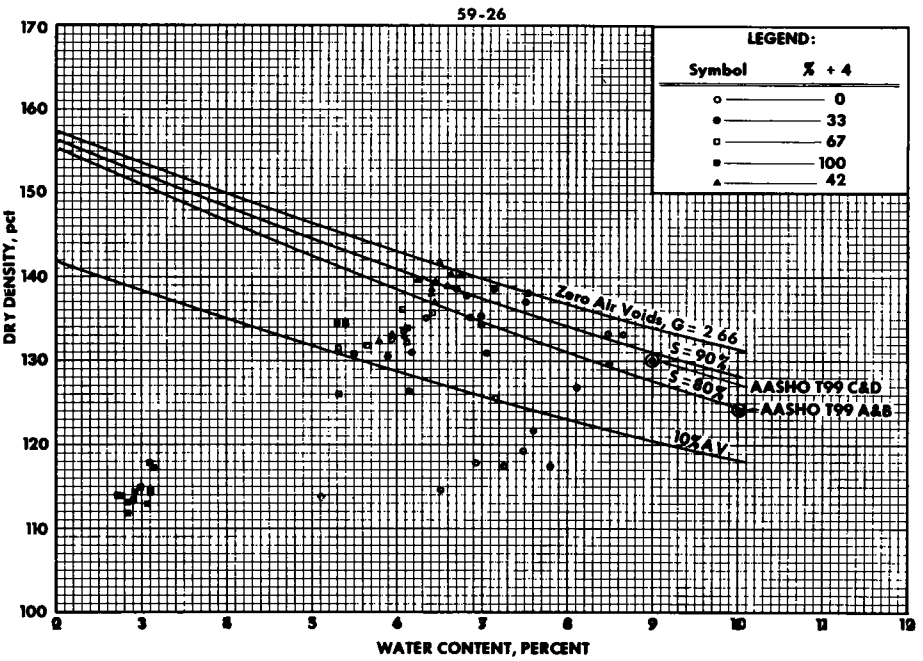

Figure 8. Laboratory test results, sample 59-26.

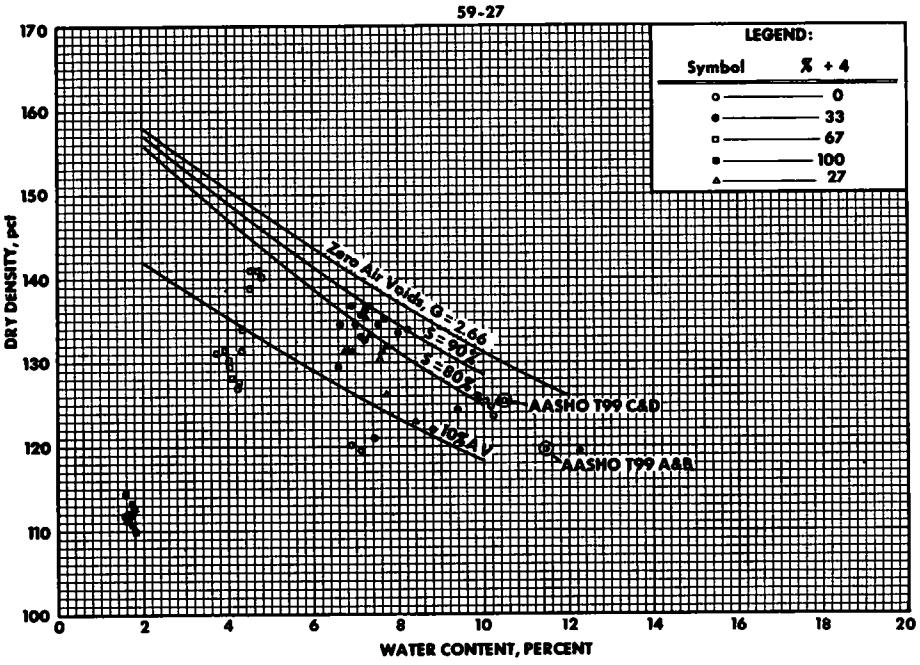

Figure 9. Laboratory test results, sample 59-27.

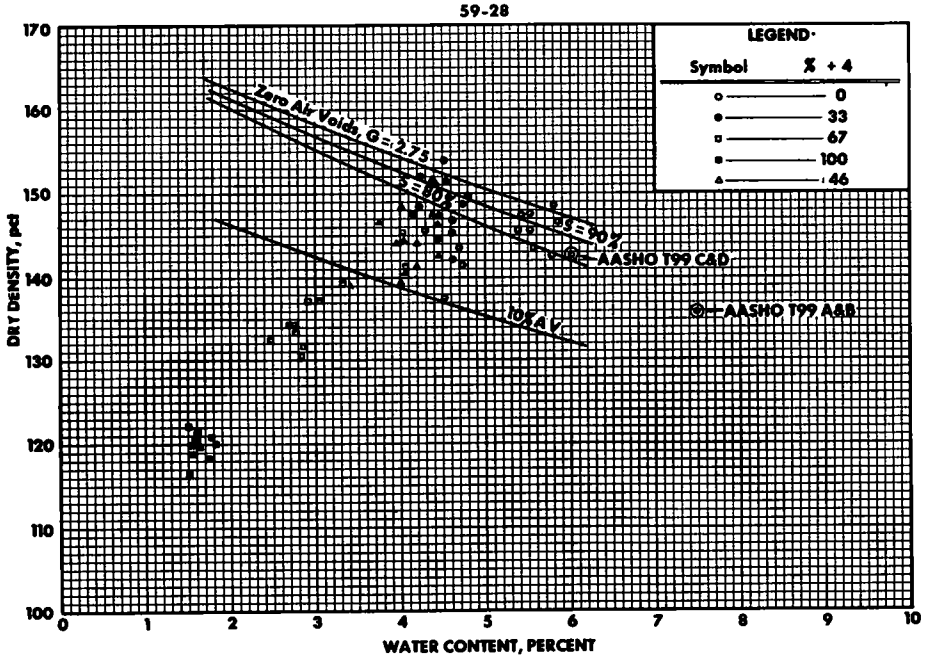

Figure 10. Laboratory test results, sample 59-28.

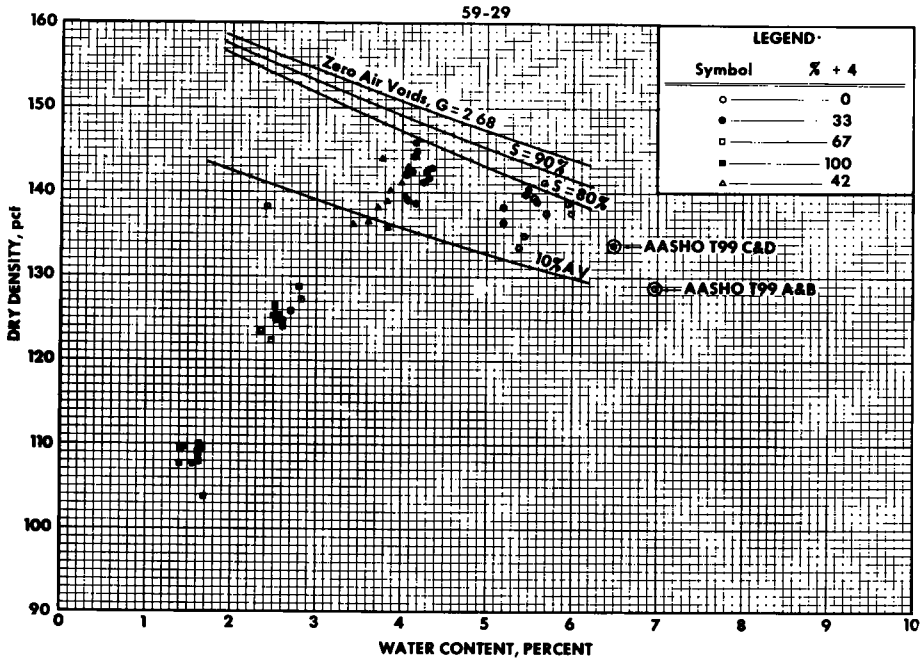

Figure 11. Laboratory test results, sample 59-29.

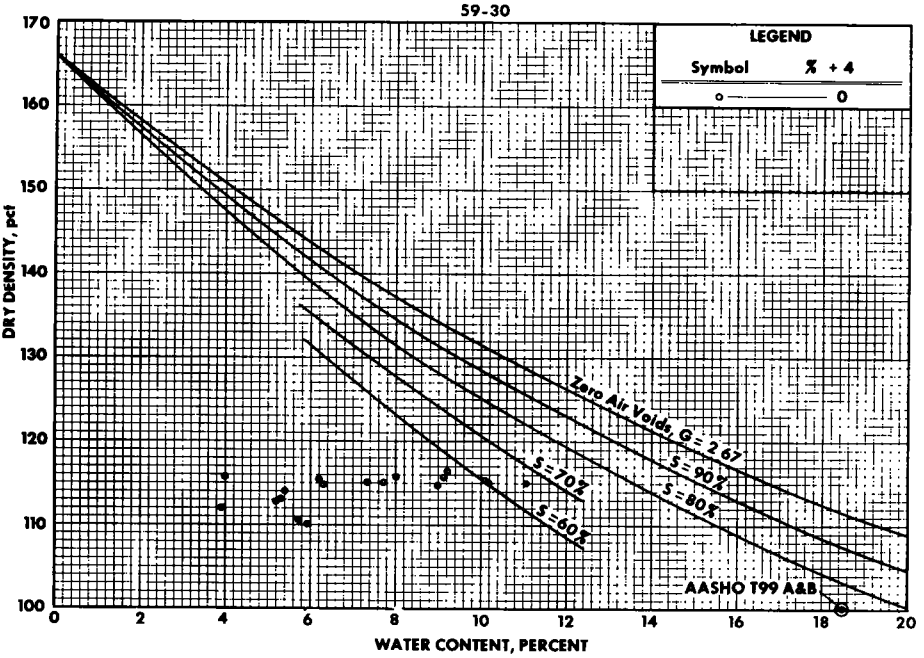

Figure 12. Laboratory test results, sample 59-30.

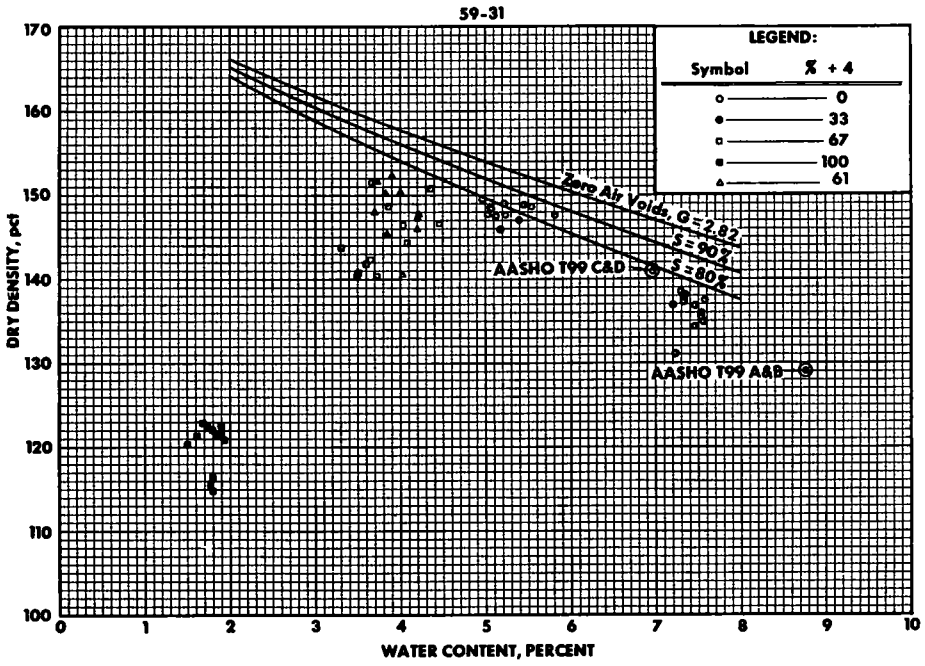

Figure 13. Laboratory test results, sample 59-31.

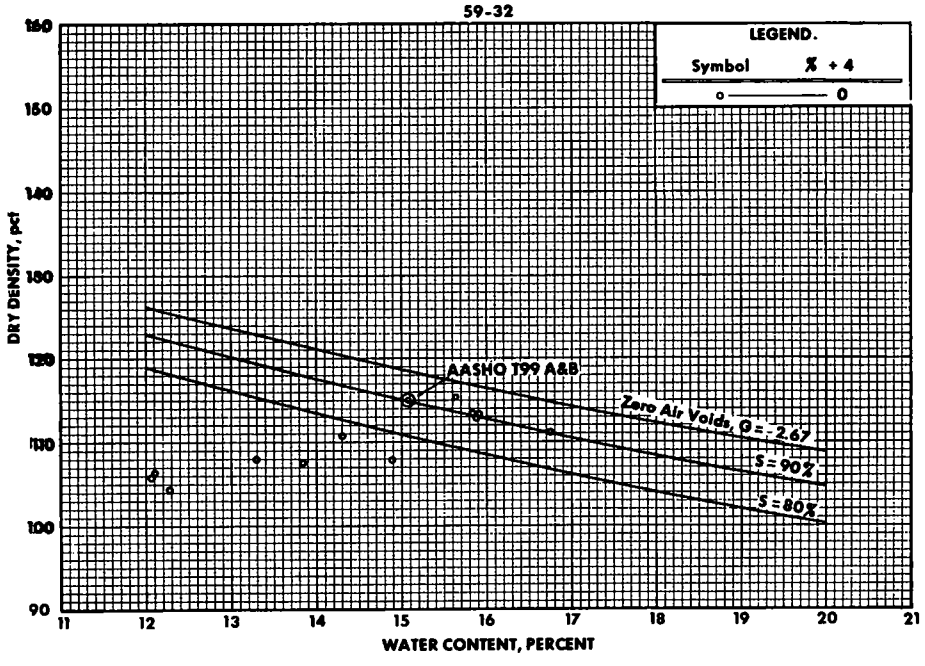

Figure 14. Laboratory test results, sample 59-32.

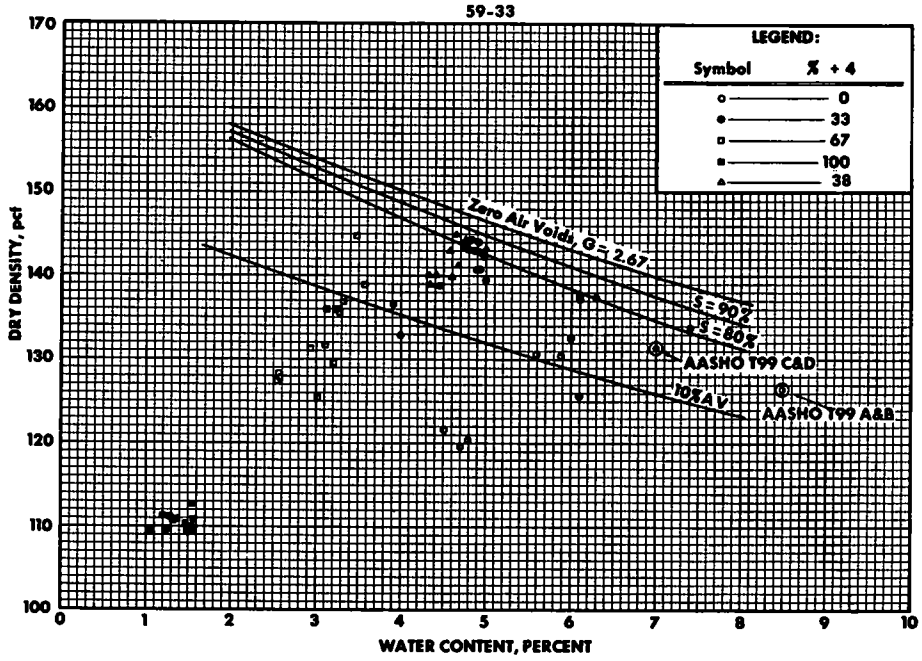

Figure 15. Laboratory test results, sample 59-33.

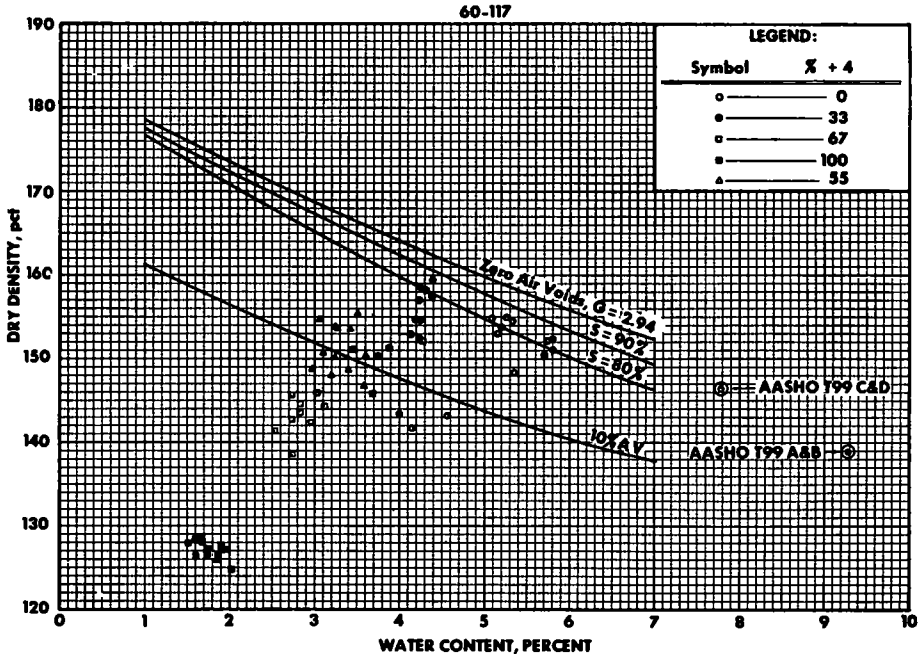

Figure 16. Laboratory test results, sample 60-117.

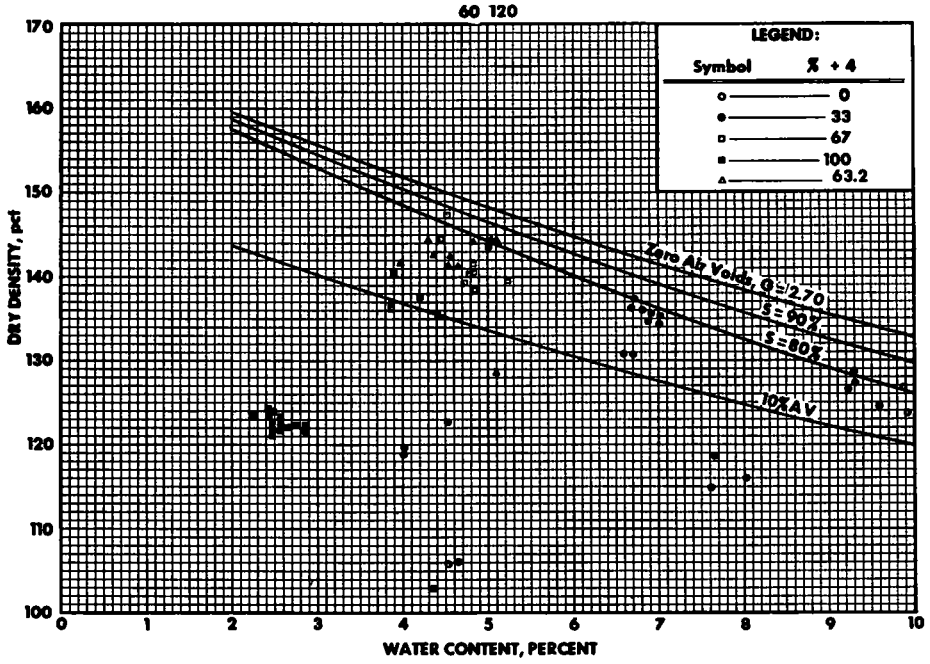

Figure 17. Laboratory test results, sample 60-120.

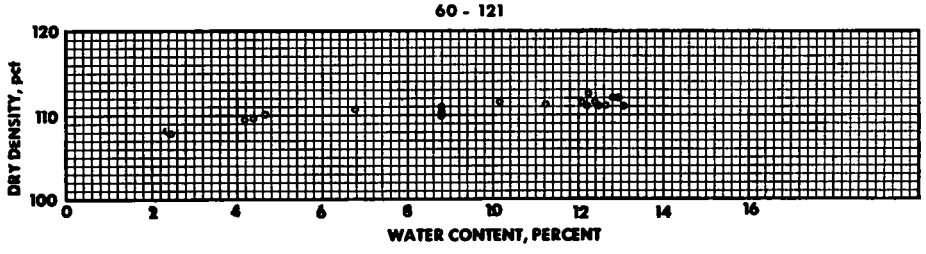

Figure 18. Laboratory test results, sample 60-121.

The 60 tests were performed in a randomized order to reduce order effects, and a standard sample was tested at intervals during the study as a check on the control.

The minus No. 4 gradation was constant, as received, and the plus No. 4 minus $^{3}/_{4}$-in. fraction was composed in each case of 50 percent minus $^{3}/_{4}$-in. plus $^{3}/_{8}$-in. and 50 percent minus $^{3}/_{8}$-in. plus No. 4. Time limitations precluded detailed study of gradation variables other than these.

Results of the tests are shown in Figures 1 through 18. From the original data, Figures 19 through 33 were developed and show the variation in density with +4 fraction. Results of AASHO T-99-A and -C tests are indicated on the figures. Values of density in Figures 19 through 33 are representative high measured ones.

FIELD TESTS

The Research Council's Pavement Evaluation Section has conducted field density tests on several experimental projects in which base and subbase courses were compacted by a number of roller types. Gradation and number of passes have been varied. Data are available for three materials from both laboratory and field tests. Figures 34 through 42 show the field densities compared to the laboratory densities.

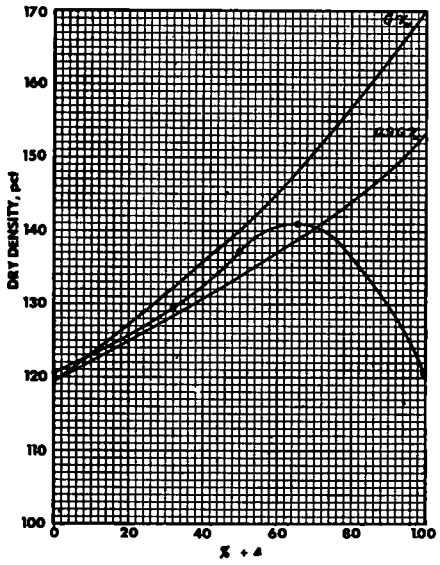

Figure 19. Laboratory density vs percent plus No. 4, sample 59-17.

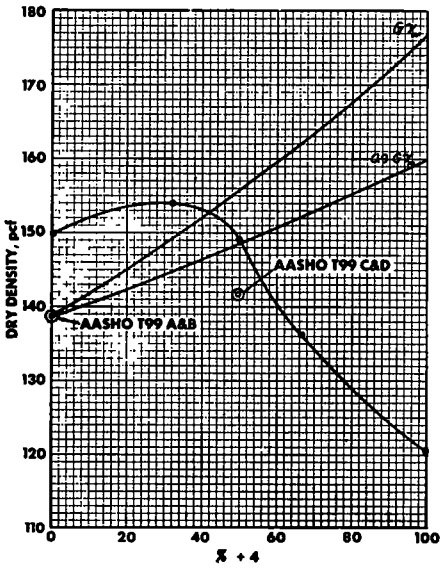

Figure 20. Laboratory density vs percent plus No. 4, sample 59-18.

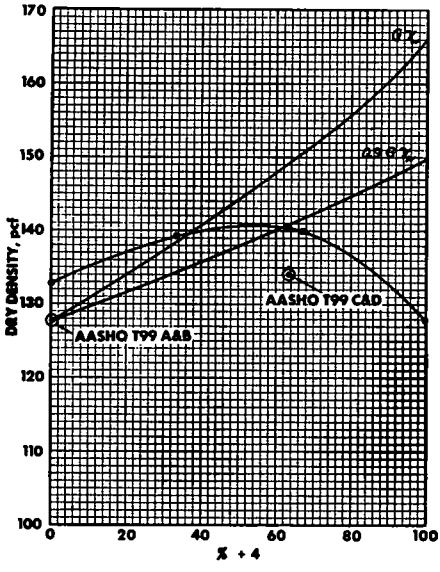

Figure 21. Laboratory density vs percent plus No. 4, sample 59-19.

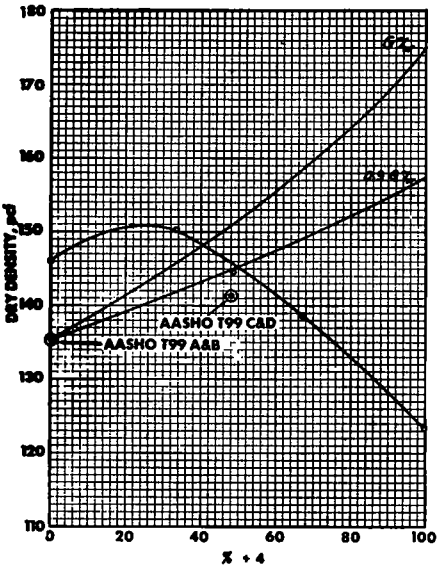

Figure 22. Laboratory density vs percent plus No. 4, sample 59-20.

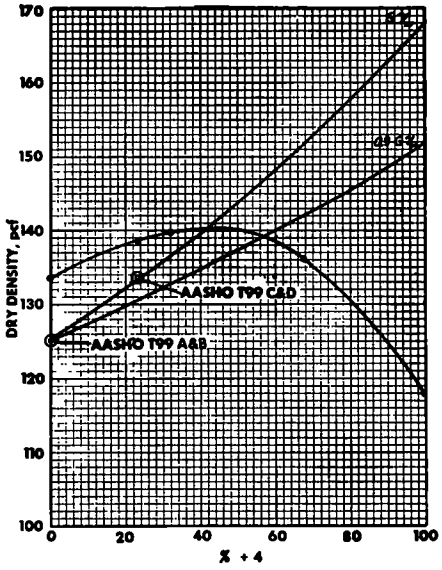

Figure 23. Laboratory density vs percent plus No. 4, sample 59-22.

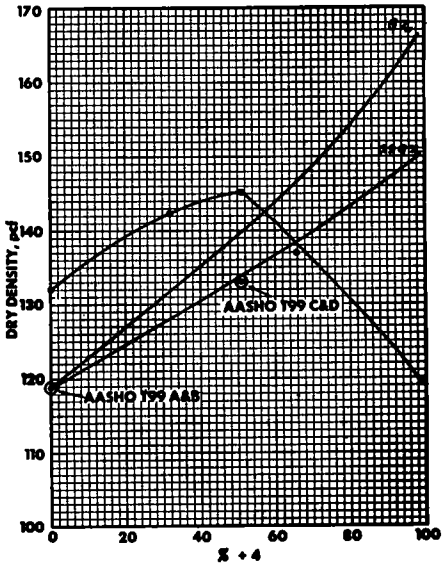

Figure 24. Laboratory density vs percent plus No. 4, sample 59-24.

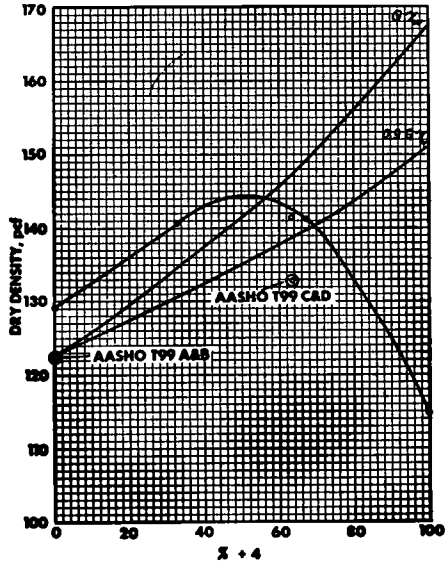

Figure 25. Laboratory density vs percent plus No. 4, sample 59-25.

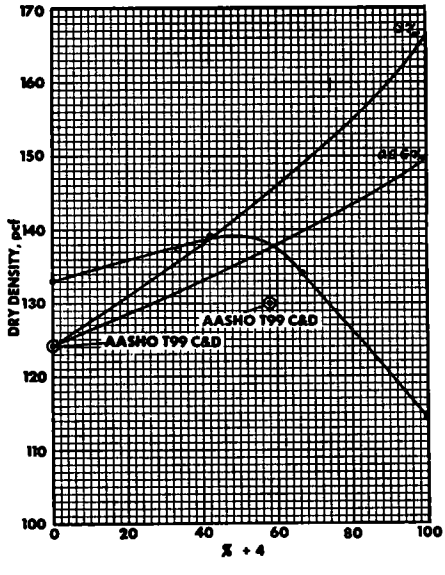

Figure 26. Laboratory density vs percent plus No. 4, sample 59-26.

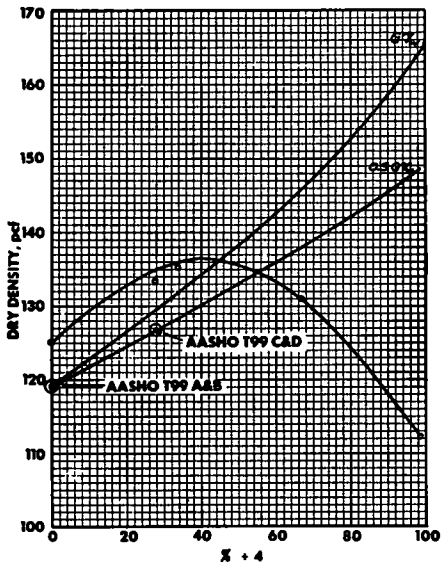

Figure 27. Laboratory density vs percent plus No. 4, sample 59-27.

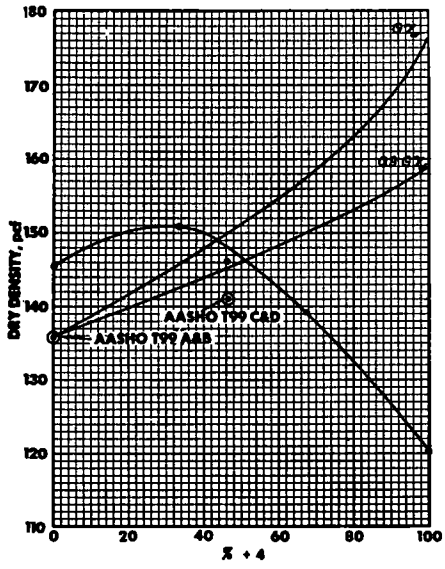

Figure 28. Laboratory density vs percent plus No. 4, sample 59-28.

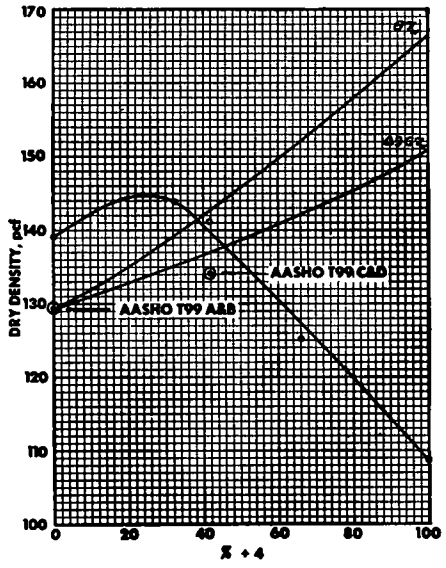

Figure 29. Laboratory density vs percent plus No. 4, sample 59-29.

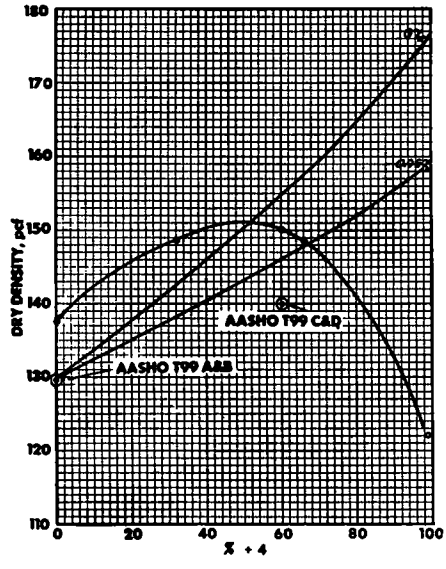

Figure 30. Laboratory density vs percent plus No. 4, sample 59-31.

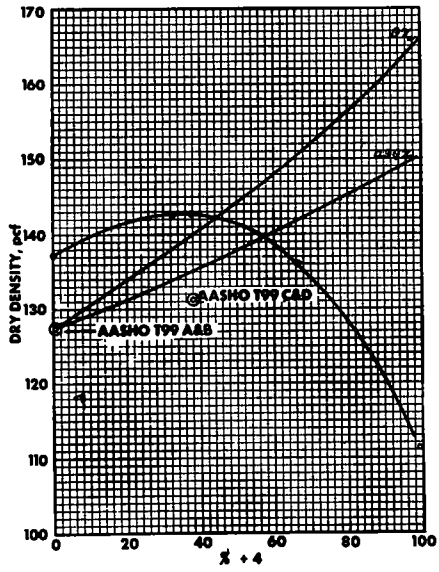

Figure 31. Laboratory density vs percent plus No. 4, sample 59-33.

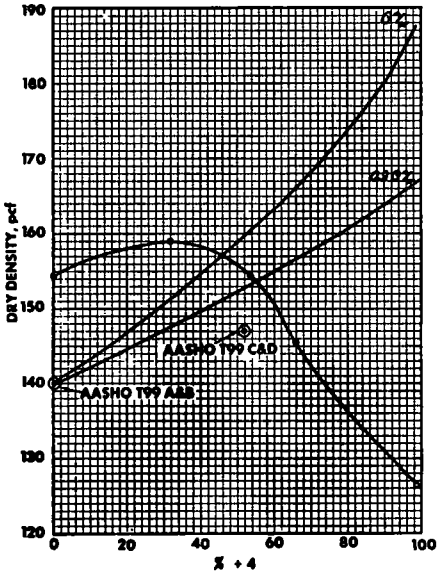

Figure 32. Laboratory density vs percent plus No. 4, sample 60-117.

CONTROL TESTS

A standard sample of crushed stone was tested at intervals during the program to assure that the testing process was still in order. The control chart is given in Figure 43.

Other tests were conducted to measure sample degradation during compaction, and this degradation was found to be on the order of 1 percentage increase on the percentage passing a given screen after 1-hr vibration and was thought to be not significant.

DISCUSSION

It is apparent that all the materials do not exhibit identical compaction characteristics. In particular, most of the compaction curves had no negative slope because water added past a certain point merely ran out of the mold during vibration; final degrees of saturation were quite variable and ranged from 60 to 98 percent.

No constant fraction of coarse material produced maximum densities; values of "optimum plus No. 4" varied significantly.

Precision of results was satisfactory; the coefficient of variation for the control sample density was only 1.7. The randomization procedures and uncontrolled variations in the gradation of the minus No. 4 fractions cause the scatter in results; however, realistic data is thus obtained and enough replication was provided to secure reliable data.

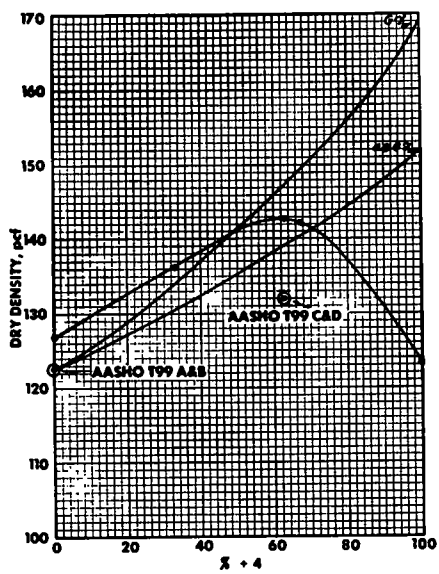

Figure 33. Laboratory density vs percent plus No. 4, sample 60-120.

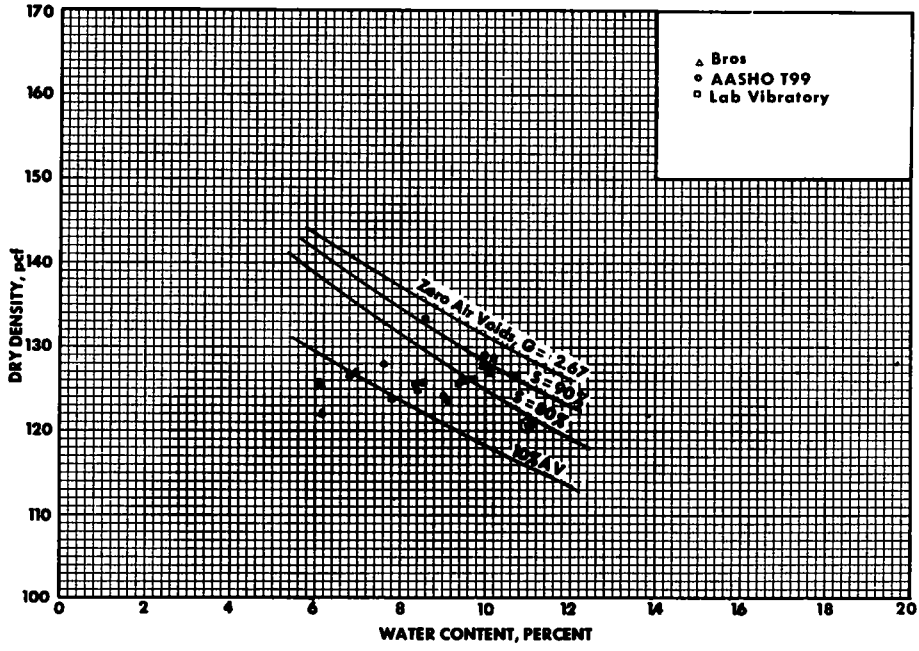

Figure 34. Laboratory and field test, sample 60-8.

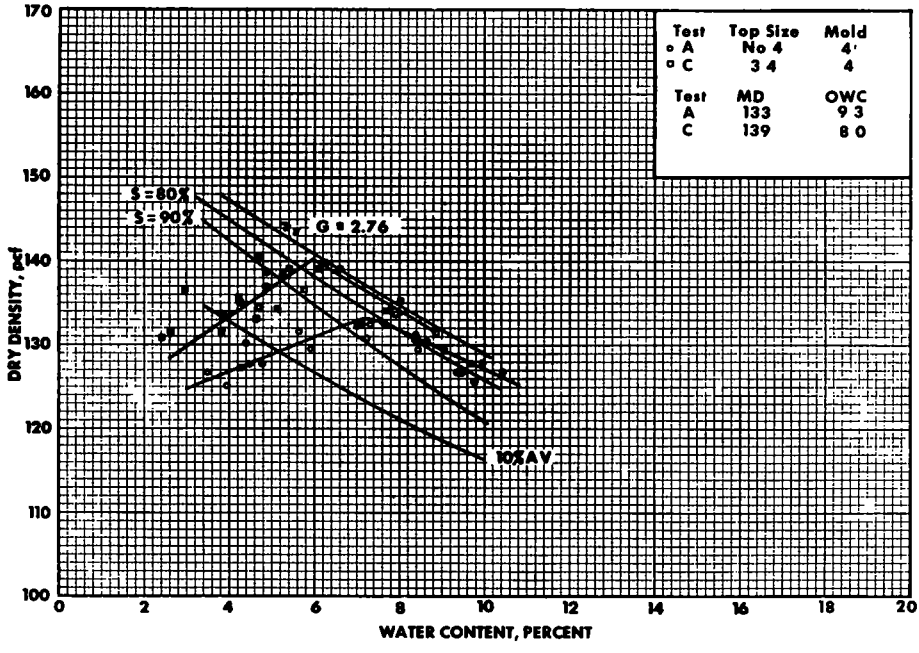

Figure 35. Standard compaction test results, sample 60-65.

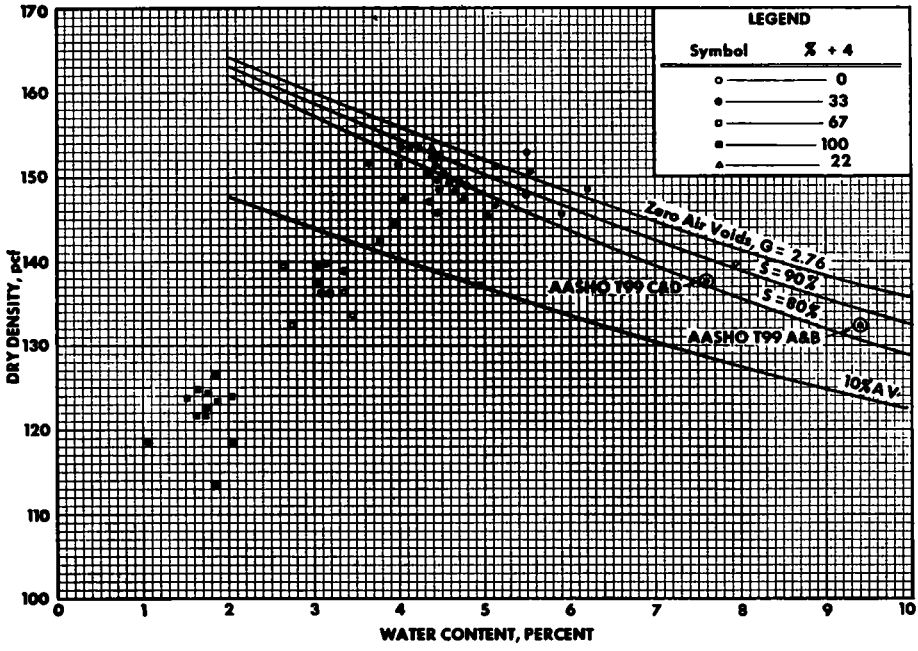

Figure 36. Laboratory compaction tests, sample 60-65.

The ratio of vibrated density to standard density was one or greater with the exception of a single material, 59-32, which was somewhat of a borderline case.

Increasing quantities of coarse material caused the expected increase, then decrease in density and the effect of water content was diminished at high plus No. 4 fractions. Neither of the correction formulas used was effective in predicting densities.

A study of the data shows no apparent typical behavior of the classes of materials except the beach sand; that is, all the crushed stone did not reach maximum density at a given gradation, and water content variation had different effects on the gravels, etc. Data was reduced to void ratios and porosities in an effort to find general comparisons, but to no avail.

Examination of the density vs percent +4 curves (Figs. 19-33) shows that the correction curves and the density curves are approximately parallel to 20 percent plus No. 4 only; this indicates the upper limit of usefulness for the correction curves.

The laboratory vibrated densities correlated well with maximum field densities in the three cases studied, as can be seen in Figures 34-42. Further work on laboratory-field correlation is scheduled.

It is apparent that current AASHO standard methods for determinations of

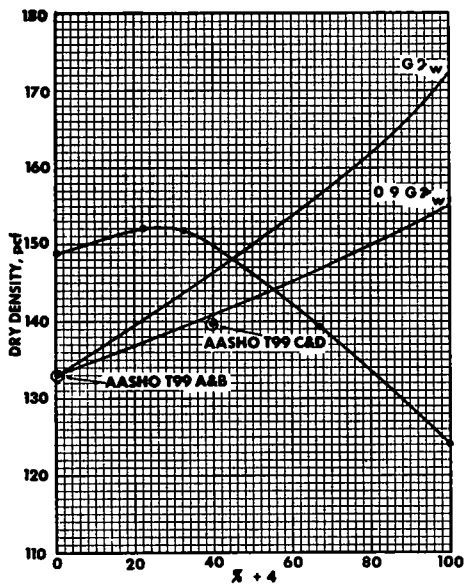

Figure 37. Density vs percent plus No. 4, sample 60-65.

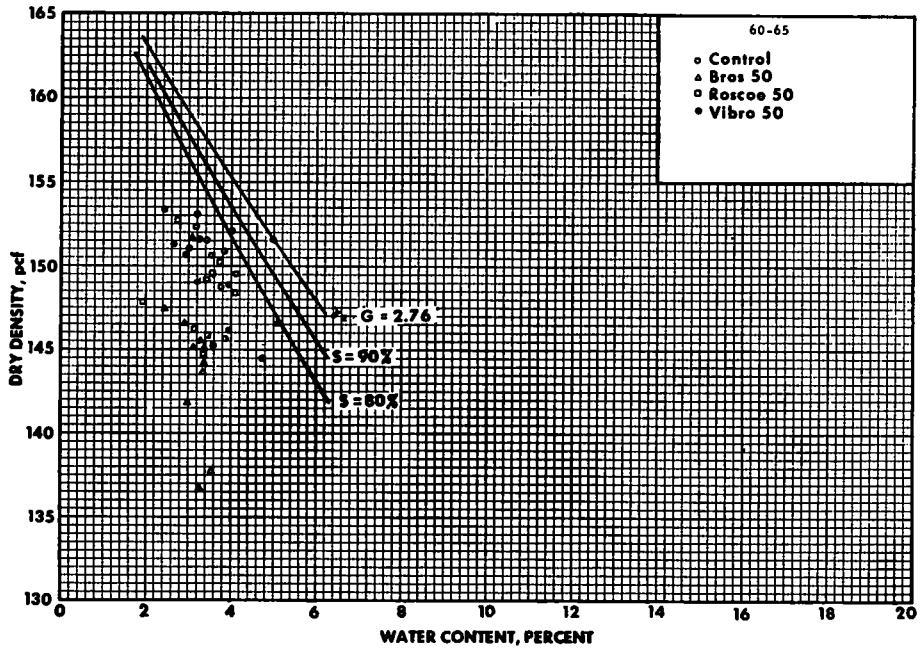

Figure 38. Field density measurements, sample 60-65.

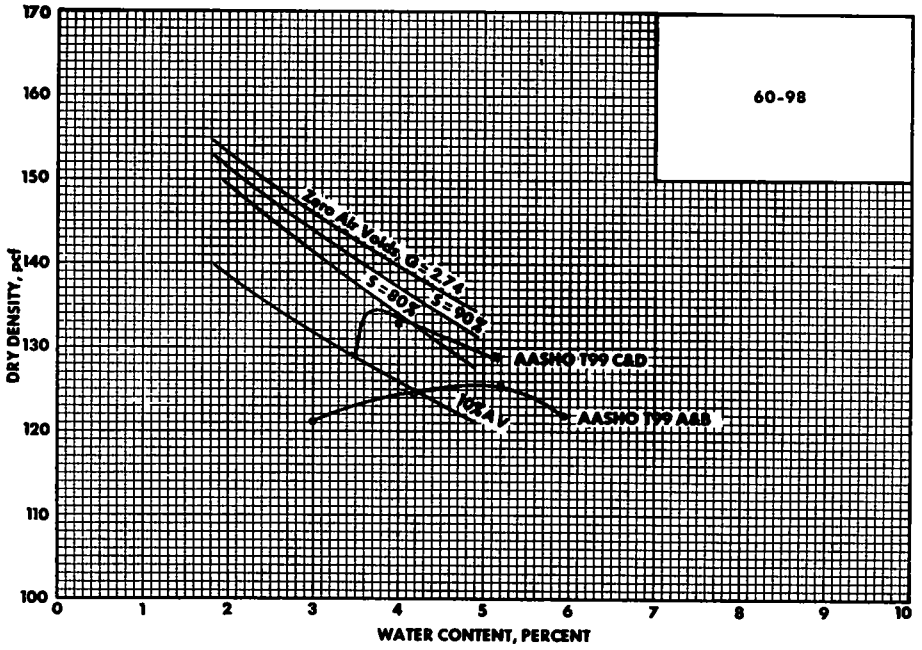

Figure 39. Standard compaction tests, sample 60-98.

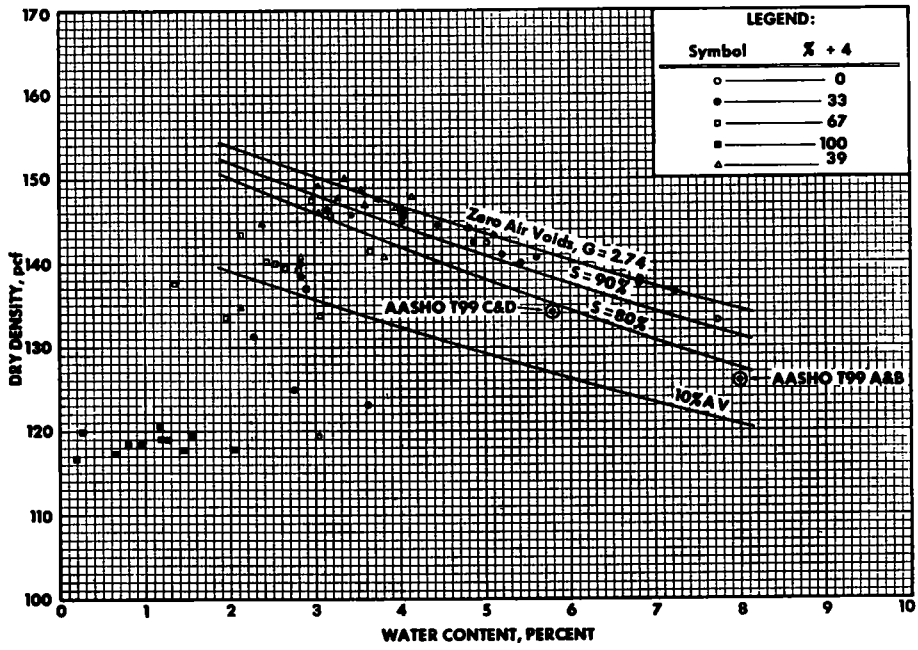

Figure 40. Laboratory compaction tests, sample 60-98.

the laboratory density of cohesionless materials and the correction methods previously discussed will not furnish realistic values of density. Recommendations from this study will advise testing coarse materials in apparatus similar to the vibratory table used in the study.

CONCLUSIONS

For the materials tested:

1. Laboratory vibratory compaction produces higher density than standard dynamic compaction and correlated well with field densities.
2. Current AASHO and ASTM standard compaction test methods give density values that can easily be exceeded by other laboratory methods and by field compaction.
3. Formulas that predict density increases caused by addition of plus No. 4 particles are likely to yield unrealistically high values, even though the formulas may be theoretically correct.

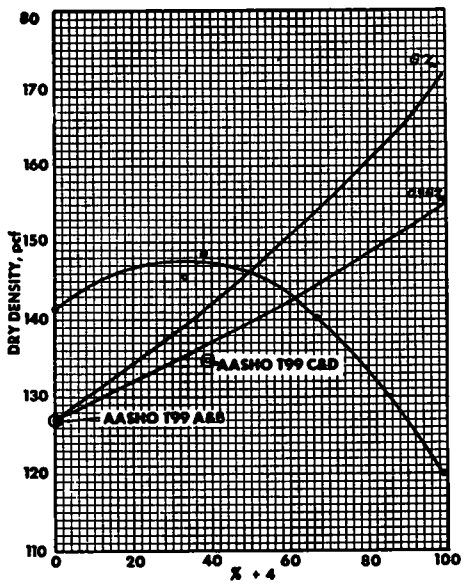

Figure 41. Density vs percent plus No. 4, sample 60-98

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1. Burmister, D.M., "Principles of Permeability Testing of Soils." ASTM Spec. Tech. Publ. 163 (1954).
2. Texas Highway Department, "Procedure THD-110." (1953).

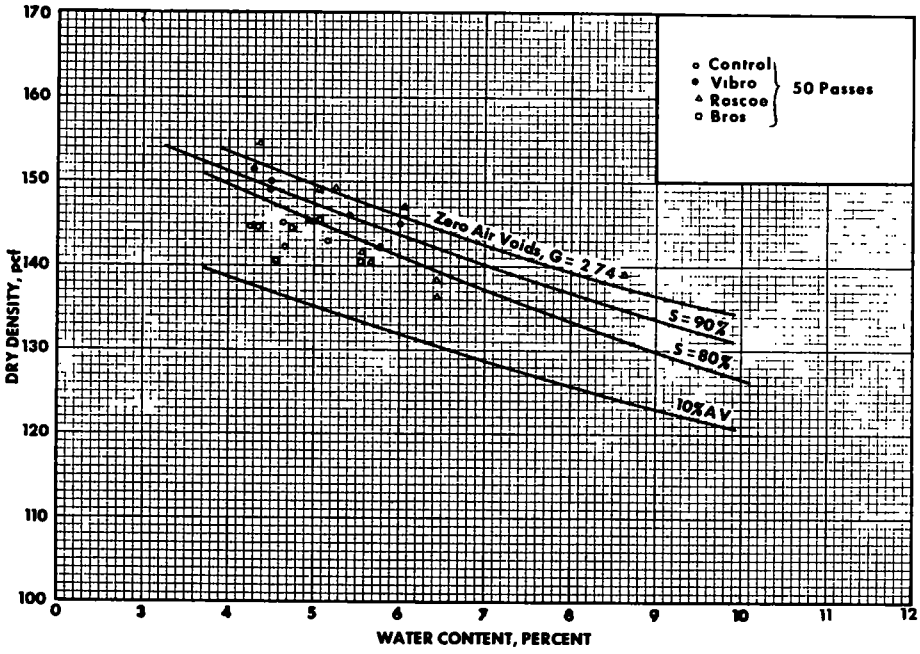

Figure 42. Field density test results, sample 60-98.

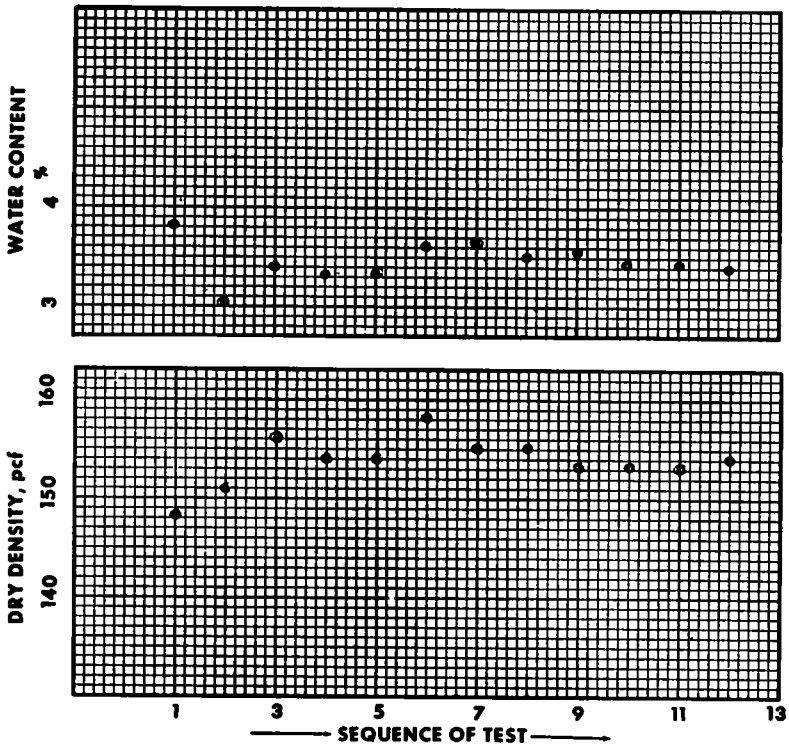

Figure 43. Control chart, sample 60-117.

3. Felt, E.J., "Laboratory Methods of Compacting Granular Soils." ASTM Spec. Tech. Publ. 239 (1958).
4. Pauls, J.T., and Goode, J.F., "Maximum Density of Noncohesive Soils and Aggregates." Procedures for Testing Soils, ASTM (1958).

Appendix

DENSITY CORRECTION FORMULA

$$D = \frac{D_f D_c}{P_f D_c + P_c D_f}$$

in which

D = corrected density, pcf;

D_f = AASHO T99 density, -4 fraction, pcf;

D_c = coarse density, (K) (62.4) (Bulk Sp. Grav.) pcf

in which

K is either 1.0 or 0.9 depending on the type of material;

P_f = fine fraction as a decimal; and

P_c = coarse fraction as a decimal.

Suggested Compaction Standards for Crushed Aggregate Materials Based on Experimental Field Rolling

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This paper describes field studies undertaken in 1960 and 1961 for the purpose of improving control over the compaction of granular base materials. A conventional vibratory roller was used to compact the base material on a number of construction projects. Short test sections were established and subjected to intensive rolling with up to 50 coverages over each test site. Density tests were made with a water balloon volumeter during various stages of the rolling operation. The data are being used in an attempt to define a mathematical or graphical expression for the maximum field density of the material relative to the percentage of coarse aggregate present.

The results indicate that for mixtures containing coarse aggregate, a number of laboratory test methods, including AASHO Method T-99, Alternates C and D, fail to produce densities nearly as high as the densities readily attainable in the field. Therefore a collateral study, to be reported separately, is under way for the purpose of developing better laboratory methods of establishing a density standard on which to base compaction control specifications.

Observations with respect to the accuracy of various methods of measuring in-place density are also included.

• THERE IS virtually universal agreement that good compaction is essential to good highway construction. Especially in the construction of flexible pavements and in the upper layers that compose the subgrade, subbase, and base courses, inadequate compaction will reduce the load bearing capacity and may lead to serious distress. If an appreciable volume of heavy truck traffic is expected to use the pavement, a definite tendency toward further densification of the layers nearest to the surface may be expected. If further appreciable densification is allowed to occur, it will result in depressions, rutting, and cracking, all of which will impair the riding quality and durability of the surface.

To combat this tendency toward densification under traffic, all layers must be compacted to adequate density during construction. The problem lies in determining what density is adequate for each of the various materials making up the flexible pavement structure. It is with a solution to this problem, particularly as it applies to granular base and subbase materials, that this paper deals.

There exists a wide variety of methods of laboratory tests for maximum density of soils and soil-aggregate materials. The type of test used by the Virginia Department of Highways for years has been the AASHO Standard T-99 test, performed on the minus No. 4 fraction only. Granular base and subbase materials, however, commonly include from 35 to 70 percent plus No. 4 aggregate, which necessitates making a correction to the laboratory density figures to compensate for the effect of the oversize particles. The formula that has been used in Virginia assumes that the minus No. 4 material is able to maintain its maximum laboratory density constant, much as though this material were a fluid, and that coarse aggregate particles merely displace a portion of the fine

fraction without introducing any additional air voids. This correction formula is derived from the basic equation (1, p. 127) which states that "the total volume of material equals the sum of the volumes of the fine fraction (including all voids) and coarse fraction (with no voids)":

$$\frac{W}{D} = \frac{P_f W}{D_f} + \frac{P_c W}{D_c} \qquad (1)$$

in which

W = oven-dry weight of whole material;
D = theoretical maximum dry density of whole material;
P_f = percentage by weight of fine fraction (expressed as a decimal);
D_f = maximum dry density of fine fraction;
P_c = percentage by weight of coarse fraction (expressed as a decimal); and
D_c = maximum or solid density of coarse fraction, 62.4 x bulk specific gravity.

If the common factor W is cancelled out, the desired term D may be found:

$$D = \frac{1}{\frac{P_f}{D_f} + \frac{P_c}{D_c}} \qquad (2)$$

If the cumbersome fraction on the right is multiplied by $\frac{D_f \times D_c}{D_f \times D_c}$, the expression may be made simpler:

$$D = \frac{D_f \times D_c}{P_f D_c + P_c D_f} \qquad (3)$$

It has been the practice in Virginia to use AASHO Standard Method T-99-57 alternate A to determine D_f, the laboratory standard density of the minus No. 4 fraction. Under the same assumptions, however, Eq. 3 may be used to correct for the presence of the oversize particles regardless of what laboratory method is used to determine D_f and regardless of the maximum aggregate size specified in this method. If, for example, the minus No. 4 fraction of a given material is found to have a maximum laboratory density of 135 pcf and the plus No. 4 fraction a bulk specific gravity of 2.64, a plot of the theoretical maximum density "D" of the entire sample for various percentages of plus No. 4 material may be prepared, by substituting the proper values in Eq. 3. This plot is shown as Curve A in Figure 1.

It is apparent, however, that as the percentage of coarse aggregate P_c increases toward 100, the assumptions on which Eq. 3 is based become unrealistic and impractical. At some value of P_c, probably in the neighborhood of 65 percent, the theoretical density "D" shown in the curve would become impossible to attain simply because there is not enough fine material to fill the voids between the coarse particles. This would be true even if the fine fraction were a frictionless fluid offering no resistance to the coarse particles in their attempt to assume an orientation for maximum density.

In a number of test methods an attempt is made to surmount this obstacle by the use of a larger top size for the test sample; alternate procedures C and D to AASHO Standard Methods T-99-57 and T-180-57 are examples in which a top size of $^3/_4$ in. is used. But if the percentage of coarse aggregate is appreciable, particle interference restricts the density that can be attained in the mold so that the resulting laboratory density will be significantly lower than that computed from Eq. 3, based on the laboratory value for the minus No. 4 fraction. A study by the Civil Aeronautics Administration (2) showed

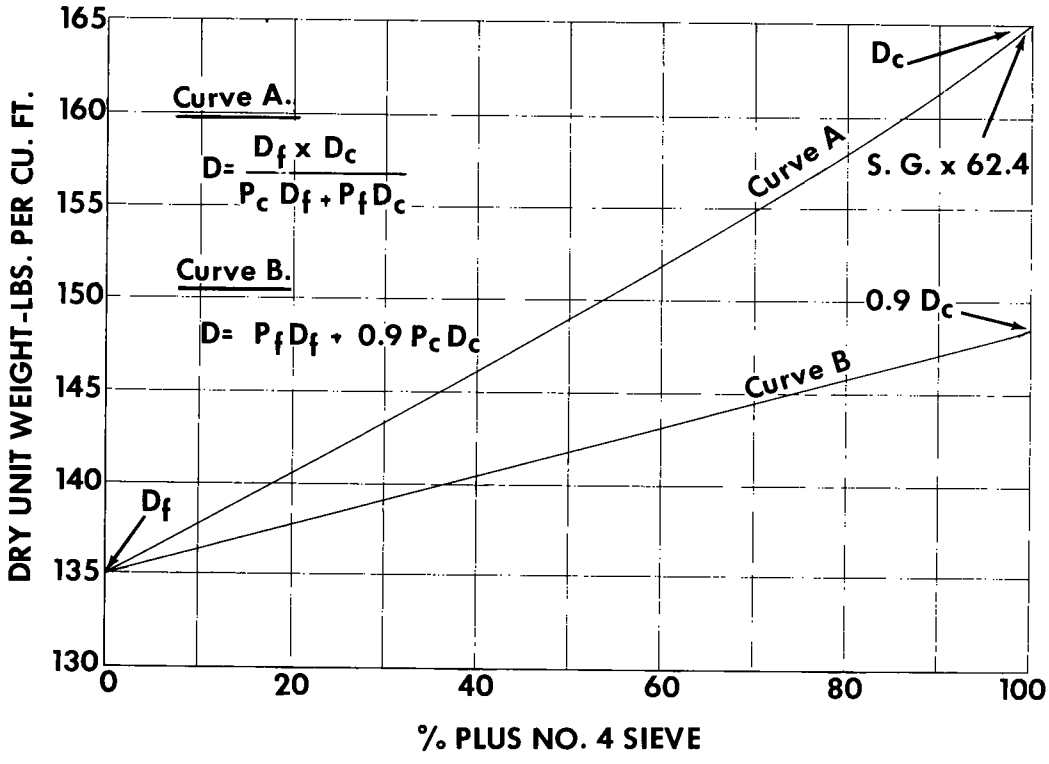

Figure 1. Typical curves for theoretical maximum density "D" in terms of D_f, P_c, and D_c.

that, up to a point, laboratory densities on samples with top size up to 1½ in. could be more closely predicted by the following expression:

$$D = P_f D_f + 0.9 P_c D_c \quad (4)$$

the symbols being the same as those in Eqs. 1, 2, and 3. The compactive effort used in the CAA study was described as "Modified AASHO" with a mold diameter of 6 in. The relationship from Eq. 4, again for a material whose minus No. 4 fraction has a maximum laboratory density of 135 pcf and whose plus No. 4 fraction has a specific gravity of 2.64, is seen graphically as Curve B in Figure 1. It is seen that regardless of how little coarse aggregate is present, its presence has the effect of lowering the molded density below that computed from Eq. 3.

Various other methods have been used by various agencies seeking to establish realistic standard densities for control over the compaction of granular materials. Some are of the impact type with the size of mold and the weight of hammer increased in the attempt to overcome the interference between the larger particles. Some are of distinctly different types; Ohio's Highway Department, for example, requires the contractor to construct a test strip that is rolled with approved equipment until no further increase in density is noted, after which all subsequent sections built of the same material are required to be compacted to at least 98 percent of the density attained in the test strip (3).

A thorough discussion of some of the variety of laboratory methods of determining maximum density and optimum moisture may be found in a paper by Hveem (4). A major point made in this paper is that many engineers have the highly erroneous impression that the terms "maximum density and optimum moisture . . . . express fundamental basic constants like the gravity constant or the boiling point of water" (3, p. 2).

This impression is obviously false; Hveem's data show the wide variety of maximum densities and optimum moisture determined on the same materials by different test methods. Some of these methods produce densities that are probably too low to be used as the basis of proper control specifications; others, perhaps, produce densities that are too high, tending to penalize the contractor unnecessarily. Therefore, in 1960, the Virginia Council of Highway Investigation and Research undertook to determine the maximum field densities attainable on a number of such materials embracing a rather wide range of percentages of coarse aggregate. From the findings of this field study it had been hoped that either a single all-purpose laboratory method or a combination of methods, with or without correction computations, might be shown as most useful in predicting these maximum field densities. One method being investigated involves the use of a vibrating table; the investigation of this method is under the direction of and is being reported separately by Chamblin (5).

It is felt, however, that as a result of the field experiments described herein, a reasonably satisfactory method of specifying density in granular materials, still based on the widely used Standard T-99, Alternate A, has been developed by the authors. The principal purpose of this paper, then, is to describe and report the results of the field experiments and to present the authors' method of specifying density.

FIELD EXPERIMENTS

Arrangements were made with the contractors on a number of field projects to permit State forces, using State-owned or leased equipment, to perform intensive rolling on short sections of the granular base or subbase materials in the attempt to compact these materials to their maximum field densities. In 1960, the experiments were performed on three projects involving four materials, using on separate sections of each a heavy pneumatic-tired roller, a lighter pneumatic-tired roller, and a light tow-type vibrating roller. On the three graded crushed stone materials, somewhat higher densities were obtained with the vibrating roller, while on the fourth, a local pit material containing only about 2 percent plus No. 4 aggregate, the highest densities were obtained with the heavy pneumatic-tired roller. In 1961 the field experiments were continued on three additional projects. Because all materials in the 1961 experiments were graded crushed stone, it was decided that only the vibrating roller would be used.

The following routine procedure was established for the installation of the test sections. Test rolling was begun as soon as possible after the material had been spread and "knocked down" by the contractor. In most cases, little or no compactive effort had been exerted before the test rolling; in some cases, however, the contractor had done some rolling, and additional compaction had been effected by construction traffic. Therefore, the initial density of the test sections was somewhat variable.

A test section, for the purposes of this study, was defined as a section approximately 300 ft long and only as wide as the width of the test roller. Five test sites were established at random locations within the central 200 feet of each test section. Preliminary density tests were made, usually at only two of the five test sites, for the purpose of determining the initial and certain intermediate densities as test rolling progressed.

Final tests to represent the maximum field density were made at each of the five test sites per section after 50 coverages with the test roller. Although this may seem to be an excessive amount of compactive effort (and indeed in 1960 when the experiment began it was not planned that this large a number of coverages would be used) the data show that the density did continue to increase on all materials beyond that attained by 30 coverages. The Appendix gives figures on the progressive densification achieved at various intervals during the rolling process; also shown in the Appendix is the surprisingly insignificant amount of degradation of the aggregate caused by fifty passes of the vibrating roller.

Conceivably, with the great variety of types of compacting equipment available today, it might have been possible to attain these maximum field densities with somewhat less effort by using a different method of compaction. However, in this study there was no intention of attempting to evaluate the relative merits of various types of compacting equipment. The main intent was to produce densities in the field that could not be appreciably exceeded with any reasonable amount of compactive effort.

The reader has probably noted that little mention has been made of the term "optimum moisture." Hveem (4) and many others have shown that optimum moisture is not a fundamental constant but varies, even for the same material, as the amount and type of compactive effort is varied. Therefore, moisture content in the field experiments was not maintained within close limits, but was varied somewhat from section to section in the attempt to find the optimum for the type of compaction actually being used. In the 1960 experiments, some of the sections were compacted on what now appears to have been the dry side. In 1961, an effort was made to put in some of the sections as wet as possible to test the theory that vibratory compaction of granular materials is best accomplished under conditions approaching inundation. In analyzing the results, however, it was found that most consistently high densities were developed in the sections whose moisture content when compacted was not more than 1 percent above or below the mean value for all sections built of that same material. Therefore, only these densities will be included in the final data reported herein. Further information on the effect of moisture content on field density will be presented in a later section.

All field density measurements were made with the aid of a Rainhart volumeter, a rubber balloon device capable of measuring test hole volumes up to 0.10 cu ft. Air pressure of 5 psi was applied to the water cylinder to make sure the balloon conformed as closely as possible to the size and shape of the test hole. This device has been tested thoroughly for accuracy and precision by various methods. Most recently, the volumes of 23 of the actual test holes in this study, which had been measured with the volumeter, were checked by making gypsum plaster casts of them and measuring the cast volumes by the water displacement method. The mean value of all these volumes as measured with the volumeter was only 1.23 percent lower than the mean of the cast volumes; the percentage divergences between the two measurements only ranged from a maximum of 1.67 percent to a minimum of 0.20 percent, the cast volume being slightly greater in all cases. From these figures, it is believed that the volumeter method combines adequate accuracy with considerably greater precision than can be obtained by the use of the conventional sand cone method. A report describing the study from which these data are quoted is available (6).

The volumes of all density test holes were kept quite close to 0.05 cu ft. This was easily accomplished by digging each hole to such a volume that the material removed just filled a ½-gal jar when lightly packed. This procedure aided in the early detection of any major errors in the values of test hole volume or weight of material removed, and is highly recommended for all types of compaction control testing.

The ½-gal jar samples from the field tests were transported to the Council's laboratory at Charlottesville for oven drying, weighing, and separating into plus and minus No. 4 fractions. After this, the materials were combined to form composite samples, coarse and fine, corresponding to each test section. The specific gravities, bulk and apparent, were determined for each composite coarse sample by AASHO Standard Method T-85. For each composite fine sample, the specific gravity was determined by AASHO Standard Method T-100 and the maximum density by AASHO Standard Method T-99, Alternate A.

This paper is based on the field and laboratory test procedures just described. All field density results can be compared with laboratory results on samples of which the field sample itself formed one part and of which all parts were obtained from this same section not over 200 ft long. The laboratory samples thus represent the field samples to a maximum degree.

ANALYSIS OF FIELD DENSITY DATA

A composite sample was made from all the jar samples taken from each group of five test sites. From the specific gravity and minus No. 4 density values obtained on each composite sample, it was possible to plot a separate Curve A (Fig. 1) defining the maximum theoretical density "D" for all variations in plus No. 4 material. The plot of this curve is greatly simplified through the use of a properly designed nomograph (such as in Fig. 2) in which Curve A becomes a straight line connecting points

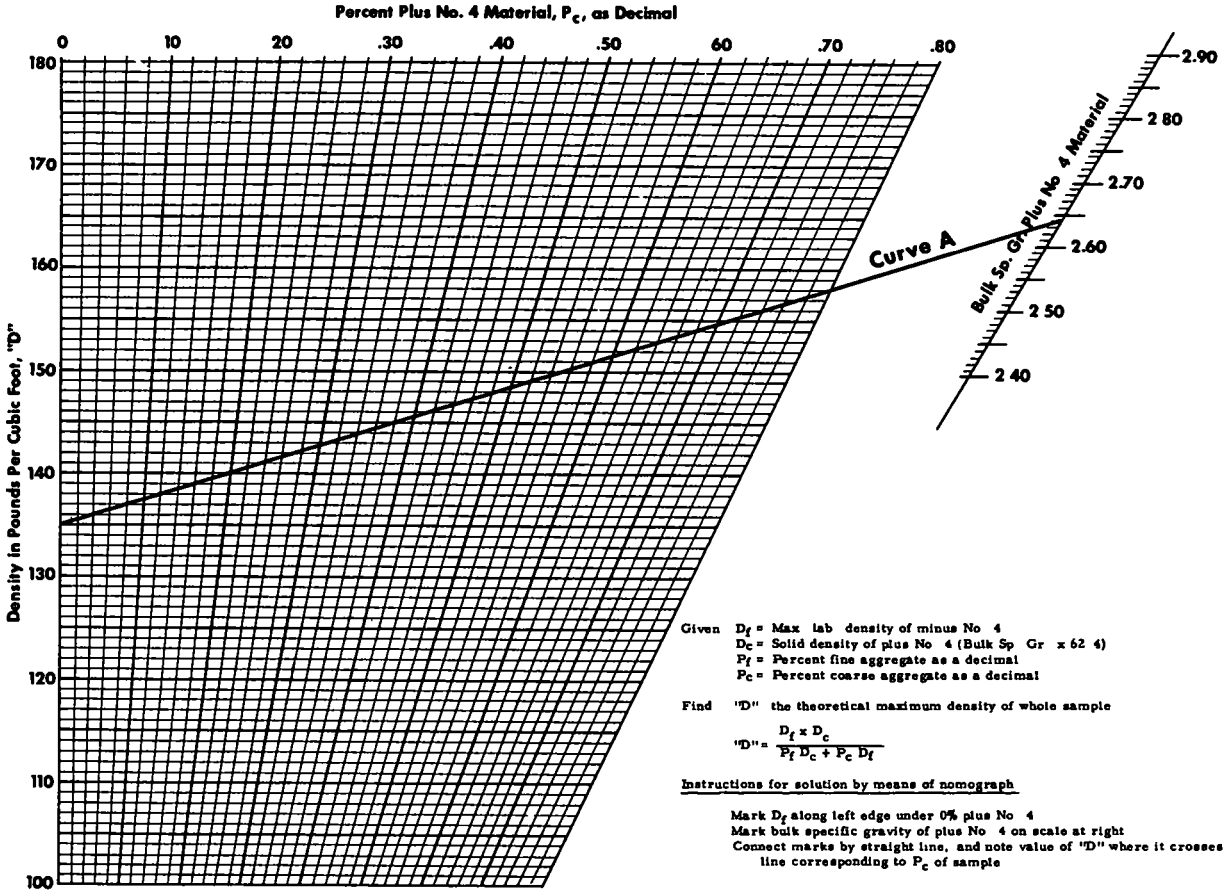

Figure 2. Nomograph for graphical solution of Eq. 3 for "D."

representing D_f on the left and the specific gravity of the coarse fraction on the right. Any point on this straight line represents the solution to Eq. 3 for the applicable percentage of plus No. 4 material. Such a nomograph was prepared for each composite sample, so that each field density determination could be expressed in terms of the same common denominator, the maximum theoretical density "D".

With all maximum field densities for all materials expressed in comparable terminology, it then became possible to prepare a scatter diagram showing the relationship between final percent of "D" and percent plus No. 4. Figure 3 shows all 108 final tests results plotted in this manner.

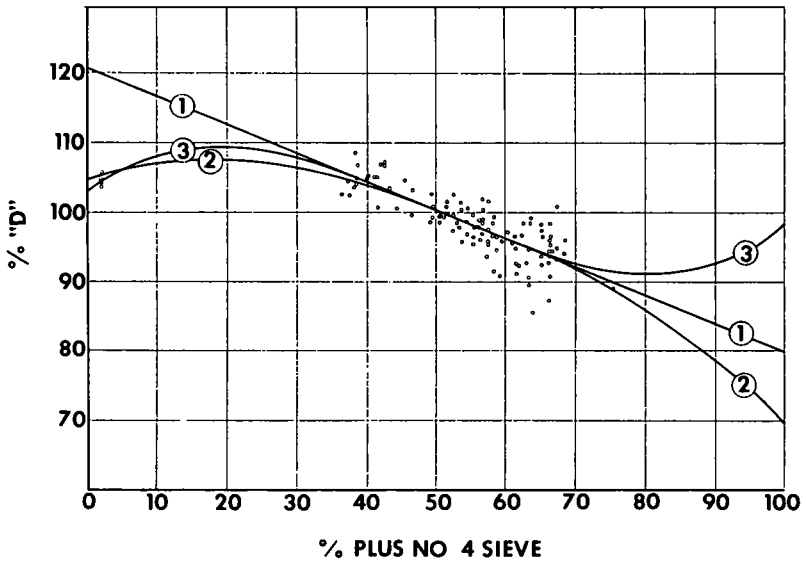

Figure 3. Scatter diagram, final field densities as percent of "D," with 1st, 2nd, and 3rd degree regression lines.

Figure 3 also shows curves of regression plotted from various polynomial equations evolved from analysis of the data by means of the IBM 1620 computer at the Central Highway Office at Richmond. These regression curves were developed in the quest for a simple workable expression for maximum field density on which specifications could be based. Curve 1 represents the best first degree equation (straight line) that could be drawn through the 102 points from the six crushed aggregate materials. Curves 2 and 3 represent the best second and third degree equations that could be drawn through the 108 points representing all the data. There was practically no difference between the standard errors of estimate for the third degree equation, plotted as line 3, and a fourth degree equation that was developed but not plotted.

It is apparent, however, that although line 3 represents a good fit with the data obtained in this limited study it would not fit points that might have been developed had the study included materials with either 10 to 30 percent plus No. 4 or more than 70 percent plus No. 4. However, between about 40 and 60 percent plus No. 4, any one of the three regression lines fits the plotted values about as well as any other, and a straight line is the simplest form to work with.

It was decided therefore to try two straight lines, the first running parallel to the X-axis from 0 percent plus No. 4 to its intersection with line 1 (which occurred at about 40 percent plus No. 4) and the second following the balance of line 1. Figure 4 shows this combination of straight lines superimposed on Figure 3.

As a simple comparative measure of the goodness of fit, or accuracy with which estimates of maximum field density can be made from the various regression lines in

Figure 4, the average absolute value of the deviations of all 108 points from each line was computed. It was found that the average deviation from the pair of straight lines was identical to that from the third degree line, with both values equal to 2.06 percent density.

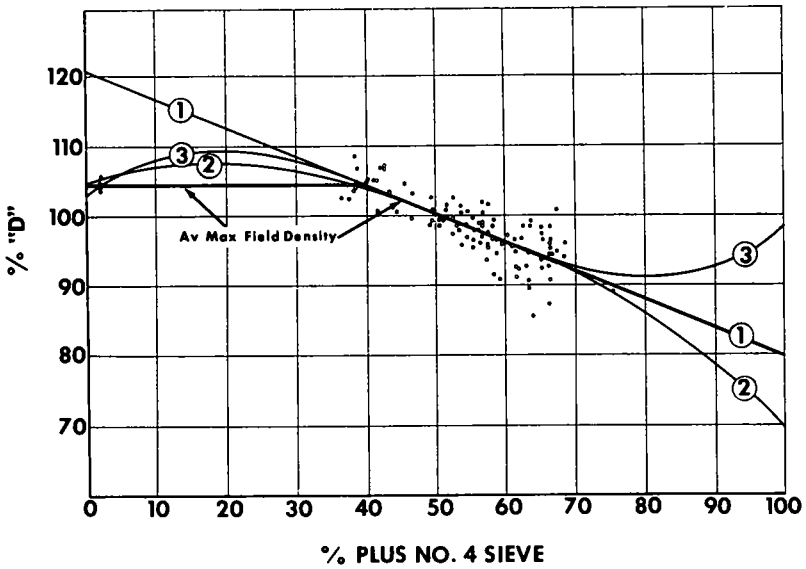

Figure 4. Same scatter diagram as in Figure 3 with straight lines superimposed to represent arbitrarily selected line of regression for average maximum field density.

As mentioned earlier, the points plotted in Figures 3 and 4 represent only those final field density measurements where the moisture content in the sample was within 1 percent of the average moisture content in all samples of the same material. It had been supposed that maximum field density in materials of this sort might be achieved at higher moisture contents, perhaps even approaching saturation. The 1961 test sections were deliberately designed to test this supposition in that each section was built on a solid, cement-treated subgrade so that compaction at high moisture contents could be accomplished without risk of softening the subgrade and producing premature failures.

It has become apparent that the supposition just mentioned is false. Figure 5 shows plots of all the final field density determinations that were not shown in Figures 3 and 4 because of moisture contents more than 1 percent above or below the average. The "average maximum field density" line in Figure 5 is the same as the one shown in Figure 4. Most of the points in Figure 5 fall below this line, indicating that there is an optimum moisture content range outside of which even 50 roller coverages were noticeably less effective in producing density. Moisture contents above the optimum range were even more detrimental than those below; final densities of those samples on the wet side were below the "maximum" line by an average of 3.0 percent, while those on the dry side averaged only 1.4 percent below this line.

RECOMMENDATION OF MODIFIED FIELD DENSITY REQUIREMENTS

The two heavy straight lines in Figure 4, then, indicate quite accurately the probable variation in average maximum density attainable in the field with respect to the percentage of plus No. 4 aggregate present. Because there is, as should be expected, considerable scatter about these lines, the specification used for compaction control should take this scatter into account. Also, it would be neither reasonable nor economical to require a contractor to produce the maximum attainable field density.

A reasonable basis for control specifications would be the requirement that all granular base and subbase materials, and certainly those placed in the top 12 in. of the

pavement structure, be compacted to an average field density that is at least 98 percent of the average maximum field density attainable. If the materials used in the field experiments described herein are properly representative, such a requirement can be described in terms of a variable percentage of the maximum theoretical density "D." For a field density sample containing a given percentage of plus No. 4 aggregate, the percentage of "D" which should be specified would be determined from Table 1.

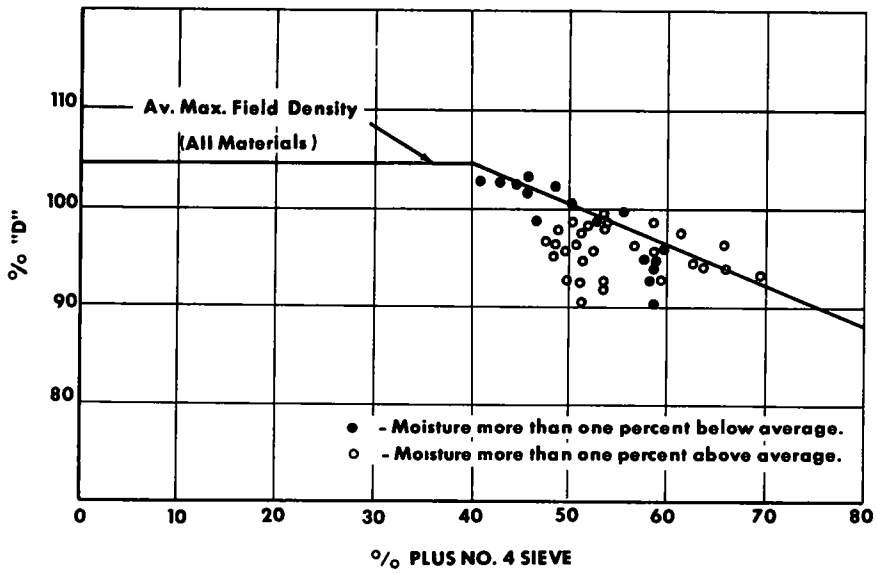

Figure 5. Final field densities on samples more than 1 percent above or below average moisture content of all samples of same material (not shown in Figs. 3 and 4).

TABLE 1

FIELD DENSITY REQUIREMENTS IN TERMS OF THEORETICAL MAXIMUM DENSITY "D"

Percent Plus No. 4	Av. Percent of "D" Required
40 or less	102.5
50	98.3
60	94.3
70	90.3
80	86.3

It may be seen that the percent of "D" required in each instance is 98 percent of the value obtained from the "maximum field density" regression line of Figure 4.

The choice of 98 percent of the average maximum field density was made in a somewhat arbitrary fashion. From the data in Table 3 (in the Appendix) it can be shown that the average density after 30 roller coverages was not always as much as 98 percent of that after 50 coverages. Therefore, if the effort equivalent to 30 roller coverages is considered excessive, this 98 percent figure may be too high. However, if control specifications are not kept high, they may fail to perform their intended function, that of insuring against subsequent densification under traffic.

The practical application of these revised density requirements may be illustrated by a typical example. Figure 6 shows again the nomograph used in Figure 2, where D_f was 135 pcf and the specific gravity of the coarse fraction was 2.64. The solid straight line connecting these points produces the solution of Eq. 3 for the value of "D" for any value of P_c. The values of "D" for 0, 40, 50, 60, 70, and 80 percent

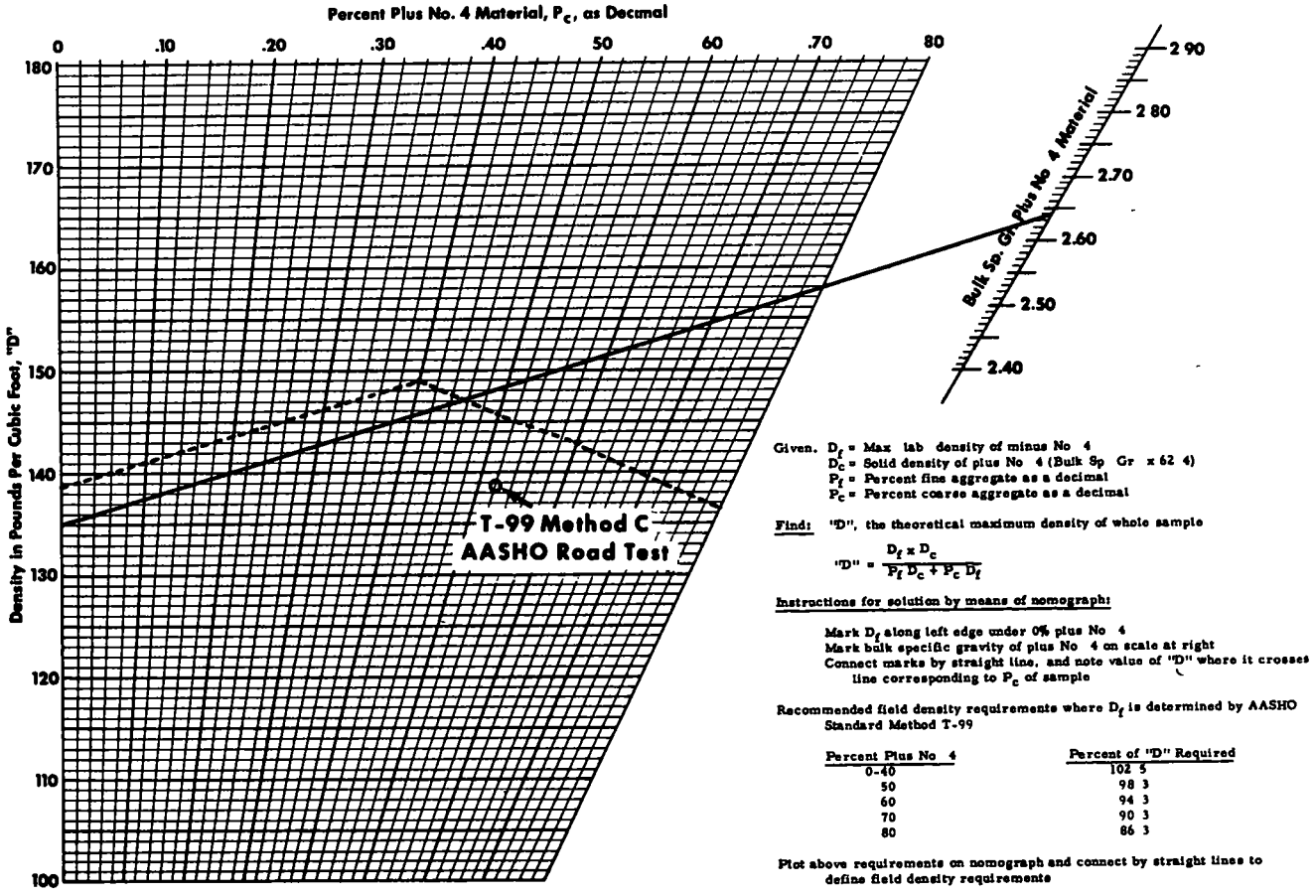

Figure 6. Nomograph from Figure 2 with suggested compaction standards added. Compaction standard used at AASHO Road Test also shown.

plus No. 4 are multiplied by the percentage corrections from Table 1, which are printed in the lower corner. These values are plotted on the nomograph and connected by straight lines. (No great error is made if only the values corresponding to p_c values of 0, 40, and 80 are plotted, because those for 50, 60, and 70 lie very close to this line.) The dotted line now represents the average field density to be required for this material for any reasonable percentage of plus No. 4. The point marked T-99 Method C will be commented on a little later.

The suggestion that compaction control be based on an average rather than an absolute minimum density requirement is made so that the authors may voice agreement with many others who feel that "a specification requiring an absolute minimum . . . . compaction is unrealistic" (7). A reasonable procedure for compaction control on a subbase or base course would seem to be one in which blocks or areas of the course in question would be laid, compacted, and tested as units, with a specified number of field density measurements made at random locations in each unit. If the average of all tests results met the requirements and if not more than one in four, one in five, or even one in ten, fell more than 2 percent below the requirement of the average, the unit as a whole would be accepted. If all these requirements were not met, the entire unit would receive additional compaction and be retested completely.

It is realized that using even the fastest of conventional field density measurement methods, a great deal of manpower is needed to make enough measurements to be sure these requirements are met. The adoption of more effective compaction control procedures will certainly be expedited by the continued development of still faster test methods. It is in the area of compaction control on granular base and subbase materials that the nuclear density measuring devices should stand their best chance of gaining general acceptance; the relative uniformity of these materials should tend to minimize the calibration problem.

PROSPECTS FOR BETTER STANDARD TEST METHODS

The foregoing analysis of field data and suggested modifications to compaction control requirements are all based on the use of a laboratory test that was originally devised for fine-grained cohesive materials, rather than for granular materials. On some granular materials, particularly those with little or no cohesiveness, it is difficult to establish a definite moisture-density relationship with the T-99 test. A more reliable test definitely would be desirable.

One thing in favor of the authors' suggested method is that it involves simple tests with readily available and portable equipment. Thus if a change in the character or gradation of the material being used becomes apparent, which does happen, a new sample can be taken and tested very quickly, even on the job if necessary.

The fact that the properties of maximum T-99 density and specific gravity do change from time to time, even on materials being produced from the same source, can be demonstrated from Table 2, which indicates the variations in materials from the same source between samples taken from different compaction test sections. In this table, the greatest variation is apparent in the results of the T-99 test for maximum density, the range in values running from 4.9 to 6.5 percent of the average values. It is conceded, however, that these variations are not entirely due to changes in the material; human error is known to account for substantial variations, and the impact test itself, as noted already, probably fails to compact some of the least cohesive samples to their true maximum density. But major variations in the material should be detectable from a simple test, and the result should be available quickly and not after the long delay often involved in sending the sample to a distant laboratory for testing by a specialized piece of equipment.

One drawback attached to the authors' method is that it fails to take account of variations in the gradation or particle shape of the coarse fraction, variations that might have a pronounced effect on the compactability of this fraction. If the compactability of the fine fraction varies between samples from the same source, it stands to reason that the compactability of the coarse fraction should vary also. This fact probably accounts for a great deal of the scatter in the data plotted in Figures 3 and 4, which is

TABLE 2

VARIATIONS IN PROPERTIES OF CRUSHED AGGREGATES

Source	Section No.	Max. Density T-99-A (pcf)	Sp. Gr. -#4 T-100	Sp. Gr. +#4 T-85
Winchester	1	139.5	2.75	2.71
	2	137.7	2.75	2.72
	3	135.8	2.74	2.70
	4	135.2	2.75	2.69
	5	135.3	2.72	2.69
	6	135.9	2.74	2.68
	7	130.8	2.76	2.71
	8	133.2	2.76	2.70
	9	134.3	2.75	2.70
	10	132.8	2.73	2.71
	11	131.7	2.74	2.70
	12	133.5	2.71	2.69
	13	134.7	2.71	2.71
	Mean	134.7	2.74	2.70
	Range	8.7	0.05	0.04
	% Range[a]	6.5	1.8	1.5
Danville (13 Sections)	Mean	123.8	2.65	2.59
	Range	7.8	0.07	0.03
	% Range[a]	6.3	2.6	1.2
Gainesville (12 Sections)	Mean	133.8	2.92	2.87
	Range	6.5	0.09	0.06
	% Range[a]	4.9	3.1	2.1

[a]Range expressed as percent of mean value.

noted to be more pronounced at the higher percentages of plus No. 4. A more completely adequate standard test method should take account of variations in compactability of both coarse and fine fractions.

In recognition that the authors' suggested method of establishing density standards in the laboratory is not without drawbacks, consideration has been given to some of the numerous other methods in use. Methods that do take account of the compactability of more than just the minus No. 4 fraction include the Humphres method (8), the alternate procedures C and D in AASHO Standard Methods T-99-57 and T-180-57, and a number of local variations.

An attempt was made to afford a comparison between the maximum field densities and laboratory densities on the same materials by some of these methods. Single composite samples of three of the materials were sent to the Washington State Highway laboratory for test by the Humphres method and for preparation of density curves. Also, tests on composite samples of four of the materials were made by Chamblin on the vibratory table (5). All materials were tested by AASHO Standard T-99 Method C.

Figures 7 through 11 show these comparisons graphically. The captions for these figures are self-explanatory, but it should be emphasized that the Humphres curves, the vibratory table curves, and the T-99-C results are for single samples only. Nevertheless, certain definite indications seem evident from these figures:

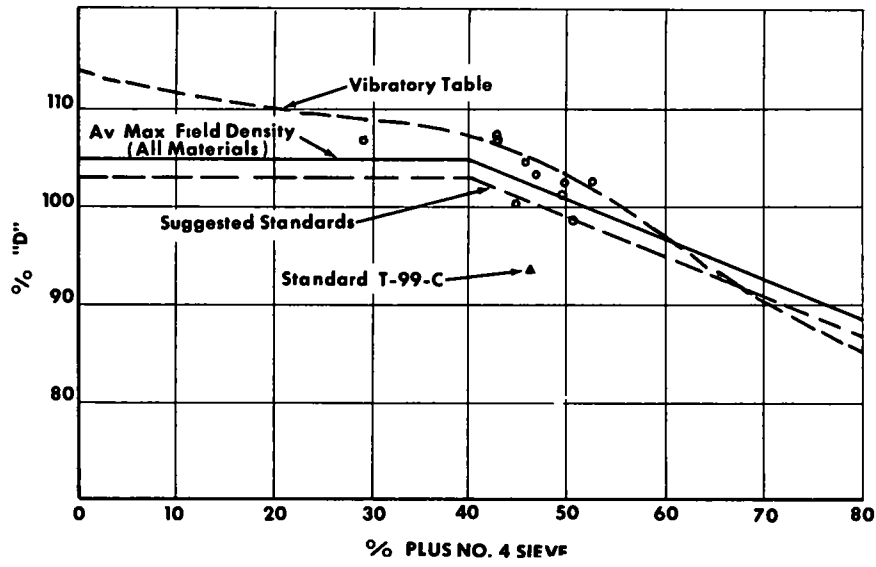

Figure 7. Final field densities, all test sections, limestone material, Madison project.

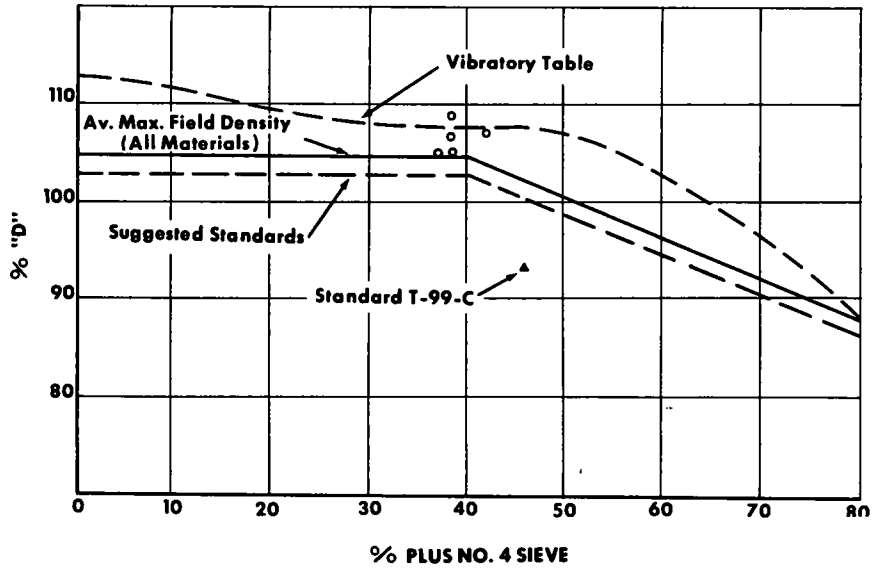

Figure 8. Final field densities, all test sections, siltstone material, Madison project.

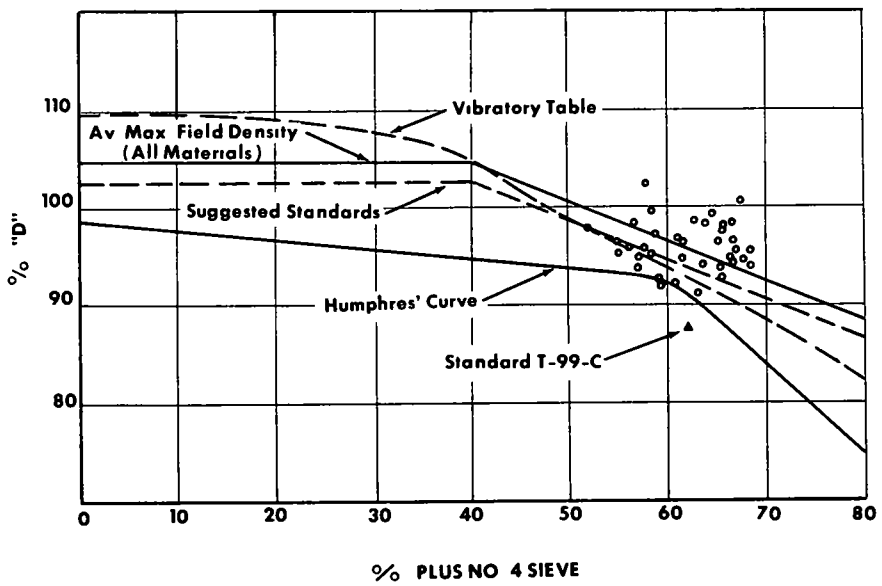

Figure 9. Final field densities, all test sections, limestone material, Winchester project.

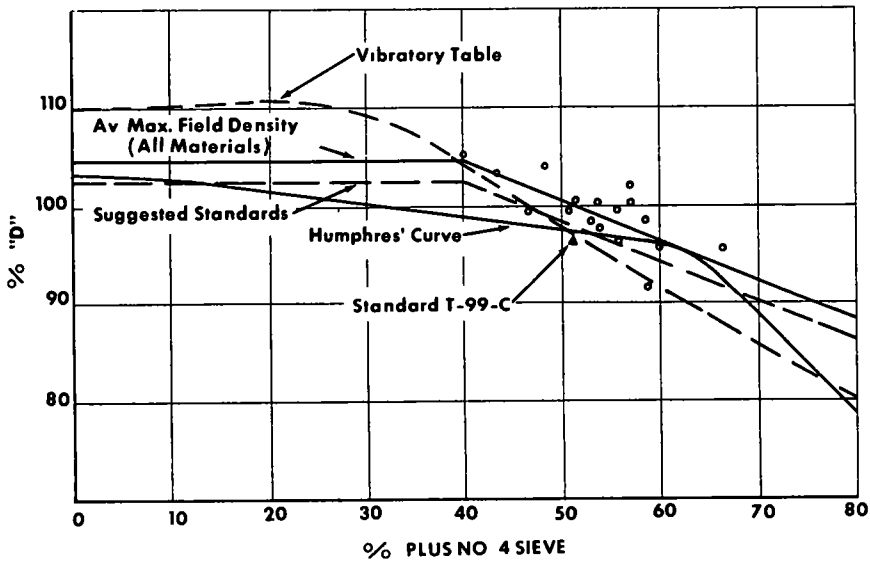

Figure 10. Final field densities, all test sections, granite material, Danville project.

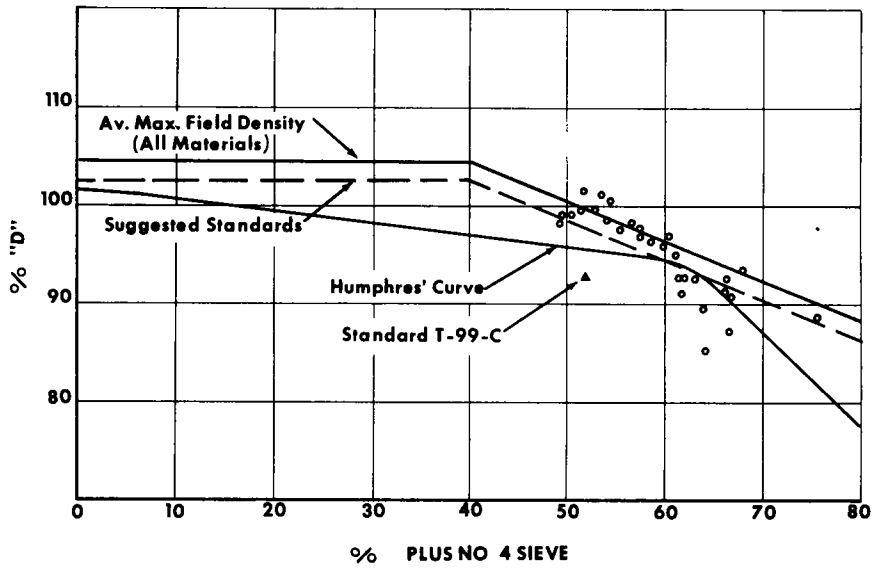

Figure 11. Final field densities, all sections, diabase material, Gainesville project.

1. The average maximum field densities are generally substantially higher than the standard density based on AASHO T-99 Method C. (A striking comparison between the present Virginia requirements, the authors' new suggested standards, and this AASHO standard is also evident from Figure 6. All values in this figure are applicable to the crushed aggregate base material used on the flexible pavement sections of the AASHO Road Test. At the reported average plus No. 4 aggregate content of 52 percent, the standard density of this material by Method T-99-C was 138 pcf. The maximum permissible density in this material at the Test Road, 145 pcf or 105 percent of the standard, approximately equals the authors' recommended requirement for average density and falls 4 pcf below Virginia's present minimum permissible density.)

2. The average maximum field densities also are generally higher than the standard established by the Humphres curve except at plus No. 4 contents in the 60 to 65 percent range. The break in the authors' curve, (based on maximum field density data) falls at a considerably lower plus No. 4 content than does the break in the Humphres curve, which was based on a number of theoretical assumptions. The apparent disagreement between the authors' field density data and the data reported by Humphres (8) is unexplained.

3. The curves produced by the vibrating table method of Chamblin bear no apparent relationship to the field densities. The vibrating table densities at plus No. 4 contents of 0 and 33 percent seem unrealistically high.

Although there are recognized drawbacks to the method of establishing compaction standards suggested by the authors, it is felt that the method provides an acceptable expedient that would result in considerable improvement over the present Virginia method based only on the maximum density computed from Eq. 3. Though the authors' standards are more rigid than those based on AASHO T-99 Method C or the Humphres curve, they would represent a general relaxation from Virginia's current requirements.

Finally in recognition of the logical complaint that the average maximum field densities may have been based on an unrealistically high compactive effort (50 coverages of the test roller), Figure 12 summarizes the results of final field density tests on a number of control sections subjected only to normal rolling by the contractor. Control sections were established on all projects, and tests were made with the Rainhart device only after compaction had been accepted by the project inspector, based on sand-cone density tests. It is noted that at a number of points the density determined by the

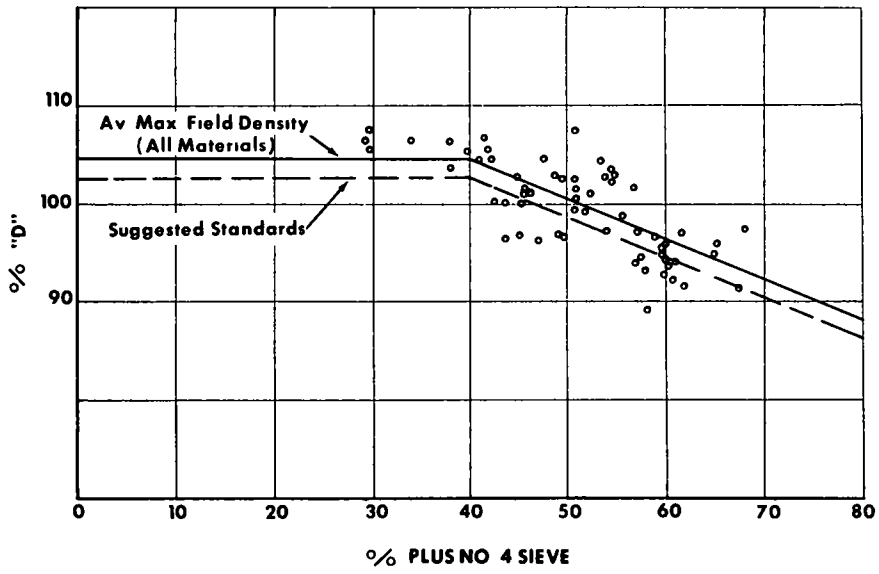

Figure 12. Final field densities, contractor's rolling only, all materials.

Rainhart method failed to meet the current Virginia requirements, but relatively few failed to meet the new standards proposed by the authors.

SUMMARY AND CONCLUSIONS

Two main points form the basis of the foregoing discussion:

1. Base and subbase materials used in flexible pavement construction must be adequately compacted to develop full load-bearing capacity and prevent subsequent further densification under traffic.
2. An essential part of compaction control is the ability to test the material in the laboratory and predict the density that, for any allowable variation in gradation, will be adequate but still attainable with reasonable effort in the field.

In recognition of these facts, the Virginia Council of Highway Investigation and Research embarked in 1960 on a joint field and laboratory study to improve control over compaction of granular materials. Test sections in the field were subjected to intensive vibratory rolling until the materials were believed to have reached their maximum attainable field density. Samples of these materials were then taken to the laboratory to be tested by various methods to see which one method or combination might most accurately predict this maximum field density.

Based on comparisons between the densities attained in the field test sections and the maximum densities achieved by certain types of laboratory test on the same materials, the following conclusions have been drawn:

1. Laboratory tests by AASHO Standard T-99 Method C, on coarse base or subbase materials, generally produce densities that are not as high as those readily attainable in the field. Control specifications based on this standard may not ensure adequate compaction to prevent subsequent densification under traffic.
2. Other nonstandard laboratory procedures, such as the vibratory table method or that developed by Humphres, do not correlate well with maximum field densities attained in this study.

By means of a regression analysis of the maximum field density data, the authors have proposed a method of establishing density standards on materials with a wide range of coarse aggregate content, based on determinations of (a) maximum laboratory density of the minus No. 4 fraction by AASHO Standard Method T-99-A, and (b) bulk specific gravity of the plus No. 4 fraction by AASHO Standard Method T-85. The results of these two determinations may be used to obtain the recommended standard density for any coarse aggregate content, as described in Figure 6.

The suggested method would establish compaction requirements considerably more rigid than those established by a number of agencies, but somewhat less rigid, generally, than those now in force in Virginia. It is felt that compaction requirements on expensive, commercially-produced base materials should be as high as economically feasible in order to develop maximum load bearing capacity. The addition of asphalt or cement to "stabilize" base materials that ought to possess adequate mechanical stability without such additives, if properly compacted, is not favored.

The foregoing density requirements are not suggested as the absolute minimum for any measurement. Such requirements have been shown to be unrealistic. A workable scheme requiring a minimum value for the average of a specific number of measurements covering a specific volume or area has been suggested.

Finally, the importance of competent technicians using reliable methods of measuring field density cannot be overemphasized. After several years of experimenting with various measurement methods, the Research Council has standardized on a method that employs the Rainhart water balloon volumeter for measurement of test hole volumes. This method has been found to possess both accuracy and precision, and is faster than other standard methods in which sand or oil is used. But no method is reliable unless it is used by a reliable technician. Every effort should be made to train these technicians properly and to impress on them the importance of their jobs. There may never be enough trained technicians with enough equipment to make all the density measurements necessary for true control over all layers of embankments, subbases, and bases. This being the case, it is highly recommended that those technicians who are available be instructed to concentrate their efforts on the upper layers composing the subgrade, subbase, and base courses.

It is the firm belief of the authors that adequate compaction control of these upper layers is not an impossible task. It is sincerely hoped that the study reported here will be of material assistance to those agencies interested in improving their compaction control for the construction of better and more economical flexible pavements.

ACKNOWLEDGMENTS

The study reported herein was not accomplished without the cooperation of a large number of individuals, both in the Department of Highways and in the contracting and equipment industries. Though so many were involved that they cannot all be named, it is hoped that all may realize the appreciation felt for their efforts.

Special mention should be made, however, of the work of R. W. Gunn, of the Highway Research Council, who had charge of the field work in 1960 and contributed greatly to the analysis of the data. Richard N. Swift also deserves mention as an able assistant in the field testing phase of the study. Laboratory work was performed by various technicians and student helpers under the supervision of B. B. Chamblin, Jr.

A special note of thanks is also due to Carl Minor, Materials and Research Engineer, Washington State Highway Department, for assistance rendered in testing the samples of crushed stone base material by the Humphres method.

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Appendix

EFFECTS OF EXTENSIVE ROLLING

Densification

The main objective of the study was to record the maximum field density obtained after extensive rolling. The observation of progression of densification was a secondary objective. When the program of research was begun it was believed by the authors that maximum density might be attainable with from 15 to 20 passes, and that little if any significant increase in density would be noted as a result of repeated coverages beyond this range. Unfortunately this was not the case. It soon became apparent that an increase in the number of coverages beyond the 15- to 20-pass range was resulting in increased density. Therefore it became necessary to set a limit as to the maximum practical number of coverages. This limit was set at 50 passes.

Time did not permit the measurement of density at intermediate intervals during the entire rolling operation on each test section. However, density measurements after 30 passes were made on three of the materials at enough locations to afford an evaluation of the effect of the final 20 passes. Table 3 presents these figures as simple averages.

TABLE 3

MEAN INCREASE IN DENSITY FROM 30 TO 50 COVERAGES

Source of Material	No. of Sites	Avg. Density (pcf)		Avg. Increase in Density (pcf)
		30 Passes	50 Passes	
Winchester	16	144.7	147.8	3.1
Danville	12	134.3	136.6	2.3
Gainesville	12	145.6	150.4	4.8

Because the increase in density caused by the final 20 passes was not at all uniform at different sites, and in fact at a few sites the density appeared to have decreased, it was decided to make standard statistical tests to determine the significance, if any, of the apparent increases. The results of these tests indicated that for each of the three materials, the increases in density due to the final 20 passes were significant. Table 4 and the calculations that follow it show how the t-test was performed on one of the three materials.

TABLE 4

DENSITY INCREASE FROM 30 TO 50 PASSES (WINCHESTER MATERIAL)

Density (pcf)		Difference, d	
50 Passes	30 Passes	(pcf)	d^2
147.4	144.0	+3.4	11.56
148.9	144.5	+4.4	19.36
142.2	146.2	- 4.0	16.00
145.0	143.8	+1.2	1.44
148.5	143.5	+5.0	25.00
148.0	149.9	- 1.9	3.61
149.5	149.0	+0.5	00.25
145.0	146.0	- 1.0	1.00
145.8	139.8	+6.0	36.00
148.8	145.2	+3.6	12.96
155.5	148.0	+7.5	56.25
149.2	140.2	+9.0	81.00
145.3	142.5	+2.8	7.84
143.3	140.1	+3.2	10.24
151.3	147.7	+3.6	12.96
150.8	144.6	+6.2	38.44
		49.50	333.91

Sample calculations:

$$s^2 = \frac{\Sigma d^2 - n\left(\frac{\Sigma d}{n}\right)^2}{n - 1}$$

in which

s = estimated standard deviation;
n = number of pairs of data = 16; and
d = difference in density at 50 and 30 passes shown in third column of Table 3.

$$s^2 = \frac{333.91 - 16\left(\frac{49.51}{16}\right)^2}{15} = 12.05$$

$$s = 3.473$$

$$t = \frac{\bar{d} - m}{s/\sqrt{n}}$$

in which

t = students test constant;

$\bar{d} = \frac{\Sigma d}{n}$;

m = specified mean value = 0; and

$t = \frac{3.09}{3.473/\sqrt{16}} = +3.556.$

The value of m is 0 because the test is employed to see if $\bar{d}$ is different from zero. From t-test tables, there is a probability of 0.05 that the absolute value of t is accidentally greater than 2.131. The calculated t-value is greater than 2.131. Therefore, a difference in density can be asserted with 95 percent confidence of being correct. The positive sign of t calculated indicates that the difference is an increase.

Degradation

To determine the amount of degradation that occurred during the 50 passes of the test roller, a sieve analysis was made on the composite samples representing the materials in their initial and final conditions (before and after test rolling). Here again it was possible to analyze the material actually used in density determinations.

Table 5 shows the mean percent passing the No. 200, No. 40, No. 10, No. 4, 3/8 in., and 1 in. sieve sizes before and after rolling, and the range of values comprising each mean. The data in this table indicate a slight tendency toward degradation.

TABLE 5

MEAN SAMPLE GRADATION BEFORE AND AFTER 50 COVERAGES

Source	Percent Passing Sieve					
	1-In.	3/8-In.	No. 4	No. 10	No. 40	No. 200
Danville:						
Before:						
Mean	89.9	61.6	45.7	35.6	24.9	12.6
Range	84-94	55-69	31-55	22-42	15-29	7-6
After:						
Mean	92.7	63.6	50.3	38.7	26.7	13.4
Range	88-96	56-72	42.59	31-49	21-35	10-17
Gainesville:						
Before:						
Mean	100	56.7	41.7	30.7	18.2	10.3
Range	100-100	48-68	33-50	24-40	12-26	5-19
After:						
Mean	100	62.4	45.4	35.2	19.7	9.8
Range	100-100	52-69	38.49	29-42	9-25	4-13

Because the increase in percent passing these various sieves was not uniform with different test sections, it was decided to apply the t-test to determine the significance, if any, of the increase in percent passing.

Table 6 and the calculations that follow it are an example of this statistical procedure.

TABLE 6

DEGRADATION ANALYSIS, DANVILLE MATERIAL (GRANITE)
Increase in Percent Passing Sieve[a]

No. 200	No. 40	No. 10	No. 4	⅜-In.	1-In.
+4	+9	+14	+15	+17	+10
-1	+1	+ 7	+ 9	+ 7	+ 8
0	-1	- 2	- 4	- 5	- 4
-1	-1	- 5	+ 1	+ 3	- 1
+1	0	- 1	0	- 4	+ 5
+3	+6	+ 9	+11	- 4	0
0	0	0	0	0	+ 2
Mean +0.9	+2.0	+3.1	+4.6	+2.0	+2.9

[a] From 0 to 50 passes.

Sample calculation t-test on No. 200 sieve:

d	d^2
+4	16
-1	1
0	0
-1	1
+1	1
+3	9
0	0
+6	28

$$s^2 = \frac{\Sigma d^2 - n\left(\frac{\Sigma d}{n}\right)^2}{n-1}$$

$$s^2 = \frac{28 - 7\,(6/7)^2}{6} = 3.81$$

$$s = 1.951$$

$$t = \frac{\bar{d} - m}{s/\sqrt{n}} \quad \frac{0.858}{0.736} = 1.165$$

in which

m = specified mean value = 0;
s = estimated standard deviation;
n = number of data points;
d = difference in percent passing;
t = students "t"; and
$\bar{d}$ = mean difference in percent passing = $\frac{\Sigma d}{n}$.

From t-test tables there is a probability of 0.05 that the absolute value of t is greater than 2.447. Calculated t is less than 2.447; therefore, the mean increase is not significantly different from 0, and there is no significant degradation apparent at the 95 percent confidence level.

Discussion

W.H. CAMPEN, Omaha Testing Laboratories, Omaha, Neb. — The authors cover two main points: (a) it is very difficult to determine maximum laboratory density in mixtures containing plus No. 4 material, and (b) field equipment can produce higher densities than is obtained by standard AASHO methods T99-57 and T180-57.

In regard to the first point, the writer's experience has shown that Eq. 3 in the paper gives high results when any amount of plus No. 4 material is used, and of course the

densities are unrealistic when the percentage of minus No. 4 is insufficient to fill the voids in the plus No. 4.

The writer has found the following procedure satisfactory for a 1 ½-in. maximum sized aggregate:

1. Prepare mixtures containing various percentages of +4 aggregate.
2. Replace all + ¾-in. aggregate with plus No. 4 minus ¾-in. aggregate prepared from the material being evaluated.
3. Run a moisture density test with each mixture using method D in AASHO T99-57 or T180-57.

In making field density tests, take samples of at least 0.10 cu ft in volume. Determine the plus No. 4 material in the sample and select the maximum density from the moisture-density curves prepared in advance.

In regard to the second point it is true that field equipment may produce higher densities than AASHO methods; however, it depends on the cohesiveness of the mixture. The field density of cohesive mixtures (even if the plasticity index is very low) can be predicted by the AASHO methods. On the other hand, cohesionless mixtures, if compacted in a wet condition by vibratory methods, can give higher results than AASHO methods. The reason is that impact laboratory methods are not suitable for compacting cohesionless mixtures. There is an urgent need for a standardized laboratory vibratory method for such materials.

Another very important related point should be brought out in this discussion. It pertains to the relationship between specified density and the density that may be produced by traffic. The writer has no specific answer to the problem but it is known that the effect of traffic depends on its weight and frequency. Densities commensurate with traffic of varying intensities will eventually be specified. Right now, the Corps of Engineer require densities of from 100 to 105 percent (AASHO 180T) for airport runways.

F.P. NICHOLS, Jr., and H.D. JAMES, Closure — Mr. Campen's comments are most welcome. With regard to his procedure of determining separate laboratory moisture-density relationships for each of several mixtures containing various percentages of plus No. 4 aggregate, the authors have two comments:

1. The procedure would involve a considerably greater amount of testing to produce a single curve of maximum density vs percent plus No. 4 than does the procedure suggested by the authors.
2. Unpublished data obtained in a Virginia laboratory study indicates that there is usually no significant difference in the maximum densities produced by Methods C and D of the AASHO standard tests. Therefore, as pointed out in the paper, the T-99 test (Method D) on materials containing appreciable plus No. 4 aggregate probably would result in standard densities too low for proper control over compaction. The T-180 test might be more suitable if it does not cause too much degradation during the course of the testing.

The desirability of having a standardized laboratory vibratory test was recognized when the study reported by Chamblin was being planned. So far, as is seen in Figures 7 through 10, the maximum densities obtained by Chamblin's method do not seem to correlate well with maximum field densities.

Finally, the authors feel that regardless of the traffic expected to use a given pavement, the more expensive base and subbase components of that pavement should certainly be given as much compaction as is economically feasible. Even if they do not densify later, the greater void content of poorly compacted mixtures invites the infiltration of water which may lead to disastrous failures, especially under severe climatic conditions. Therefore, the need for compaction standards that closely parallel the maximum densities obtainable seems self-evident.

Stabilization of Beach Sand by Vibrations

LINO GOMES, Engineer, Soil Testing Services, Inc., Chicago, Illinois, and LEROY GRAVES, Associate Professor of Civil Engineering, University of Notre Dame, Notre Dame, Indiana

• STABILIZATION of sands has been achieved by many methods, such as mechanical, chemical, addition of admixtures, grouting, and compaction. Of these methods, the most economical has been compaction, which can be achieved in many ways; for exampled, rollers, vibrotampers, and vibroflotation.

It has been reported that heavy duty pneumatic rollers imposing a pressure of about 150 psi have compacted sand to a depth of 6 ft below the ground surface. Vibrotampers, weighing 435 lb, operating at 2,100 cpm and producing a compacting force of 10,000 lb are reported to cause compaction of over 95 percent of modified AASHO on lifts up to 15 in. in one pass or two. The vibroflotation process, which imparts a centrifugal force of 10 tons at 1,800 rpm, is reported to compact sand up to a radial distance of 5 ft giving densities of 90 percent of optimum to depths in the range of piling.

Much laboratory research has been done on compaction of sands. One project conducted at the California Institute of Technology (1) by placing on the surface of a sand pit 10 ft square and 6 ft deep, an oscillator weighing 61 lb and driven at frequencies from 170 to 3,450 rpm led to the conclusion that the maximum compaction was obtained at resonant frequency involving several variables such as elastic constant of soil, vibrator dimensions, weight of vibrator, dynamic force, and base plate dimensions. Maximum density from 90 to 95 percent of Modified AASHO was obtained in a few seconds to a depth of twice the width of the oscillator.

The authors felt, after reviewing the field practice and laboratory research on the subject, that it would be profitable to investigate the compaction of sand with almost weightless tampers having several base plate dimensions and operated mechanically at the surface of dry sand at varying frequencies including the supersonic range. It was also decided to include some evaluation of the maximum possible laboratory sand density in view of the fact that though several methods have been suggested, none has been accepted so far as a standard.

APPARATUS

The compaction apparatus was constructed by attaching three aluminum plates 3 by 2 ¾ in., 4 by 2 ¾ in., and 5 by 2 ¾ in. of ⅛-in. thickness one at a time to the cone of a heavy duty loud speaker, as seen in Figure 1.

The speaker and plate were made to vibrate by an audio-oscillator augmented by an amplifier. A voltmeter across the supply line controlled the input voltage to prevent overloading the speaker. A cathode-ray oscillograph helped in the calibration of the audio-oscillator and also in the regulation of the precise frequency during the tests.

The sand to be compacted was contained in a glass-sided tank with a grid of 1-in. squares painted on one side. This tank was placed on a three-legged jack to permit raising and lowering of the tank during the compaction process. The complete apparatus is shown in Figure 2.

The dry sand chosen for the investigation has a uniformity coefficient of 4.35 and a grain-size distribution as shown in Figure 3. According to Hough (2), "an ordinary beach sand 'processed' to some extent by wave wash would have a (uniformity) coefficient of about 2 to 6." The selected grading thus has the uniformity coefficient of beach sand and in addition it fits within the grading limits for a well graded sand as specified by AASHO, M6-51, (3) as shown in Figure 3.

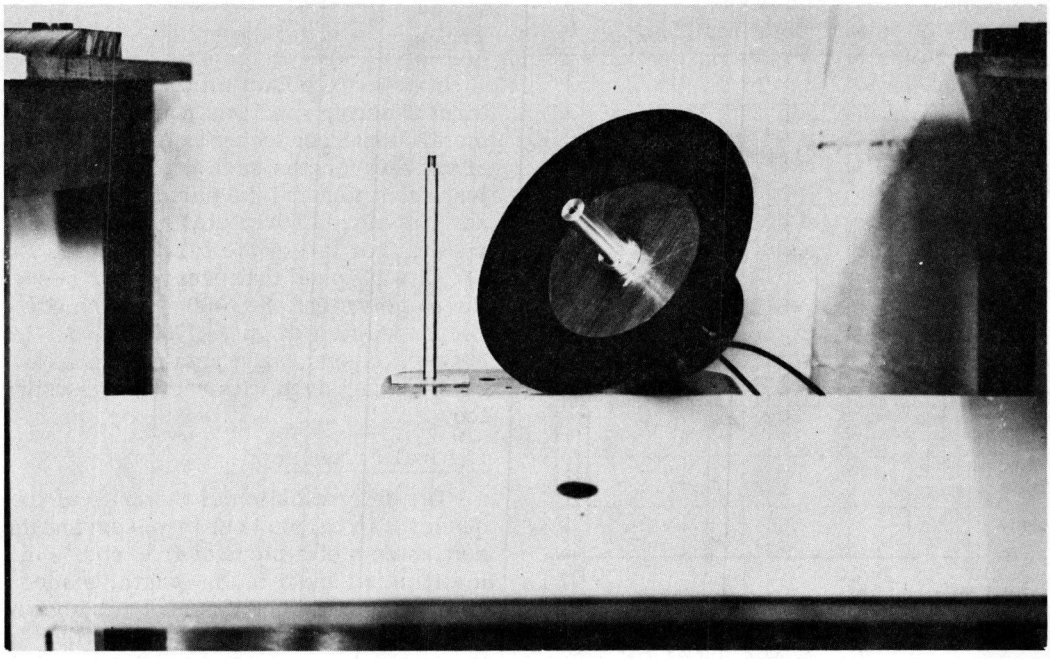

Figure 1. Details of speaker and tamper.

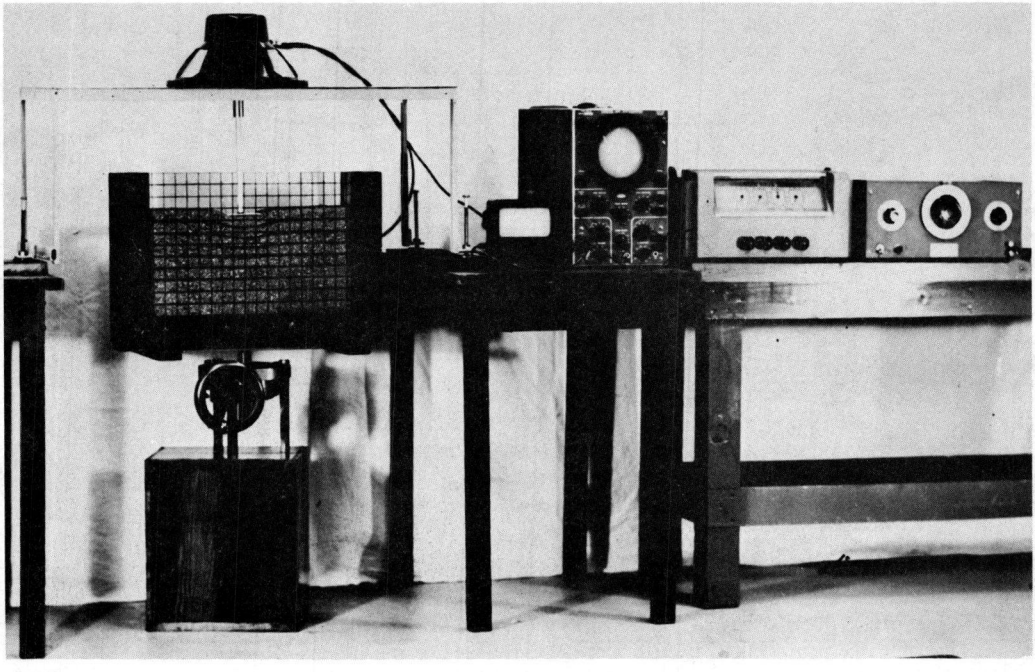

Figure 2. General view of apparatus.

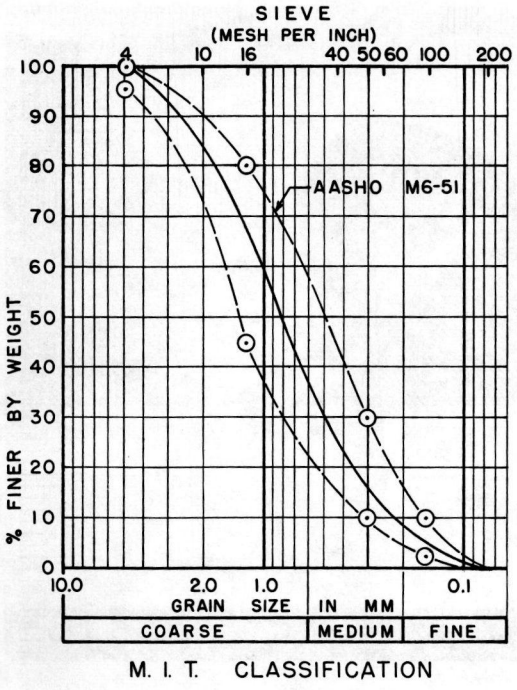

Figure 3. Mechanical analysis of sand.

PROCEDURE

Placing of Sand

In order to obtain minimum and uniform density, sand was poured through a metal funnel connected to extension tubes of varying lengths such that the extension just about touched the surface. The funnel was moved horizontally without giving rise to free fall of the particles (see Fig. 4). It was found that every layer needed the same weight of 1,400 g, which corresponded to a density of 102.5 pcf. In between layers, sand retained on a No. 60 sieve and dyed with red tint was sprinkled.

Critical Frequency

The determination of the critical frequency was carried out by observing the settlements of a piece of iron rod ½ in. in diameter and 7 in. in length, placed vertically on the sand surface as shown in Figure 5. A dial gage measured the settlement. The entire range of frequencies from 18 to 20,000 cps was tried with a duration of ½ min each time, taking care to see that the sand density was 102.5 pcf before each trial. Because the process of placing the oven-dried sand without segregation of sizes and with uniform minimum density was laborous and time consuming the entire range from 18 to 20,000 cps was investigated by using a 3-in. tamper only. However, within the range that gave

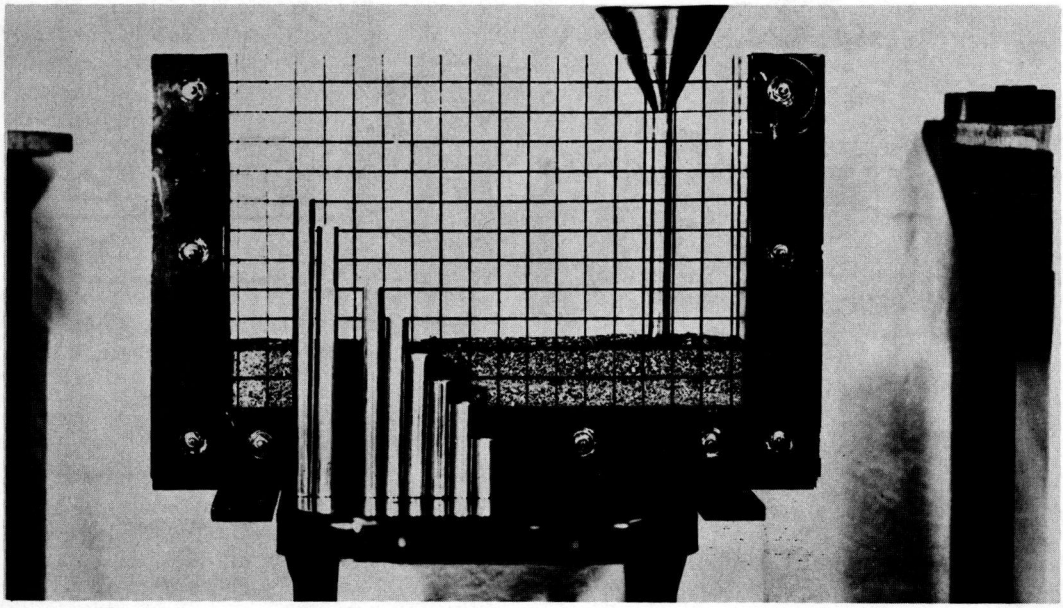

Figure 4. Apparatus for placing sand.

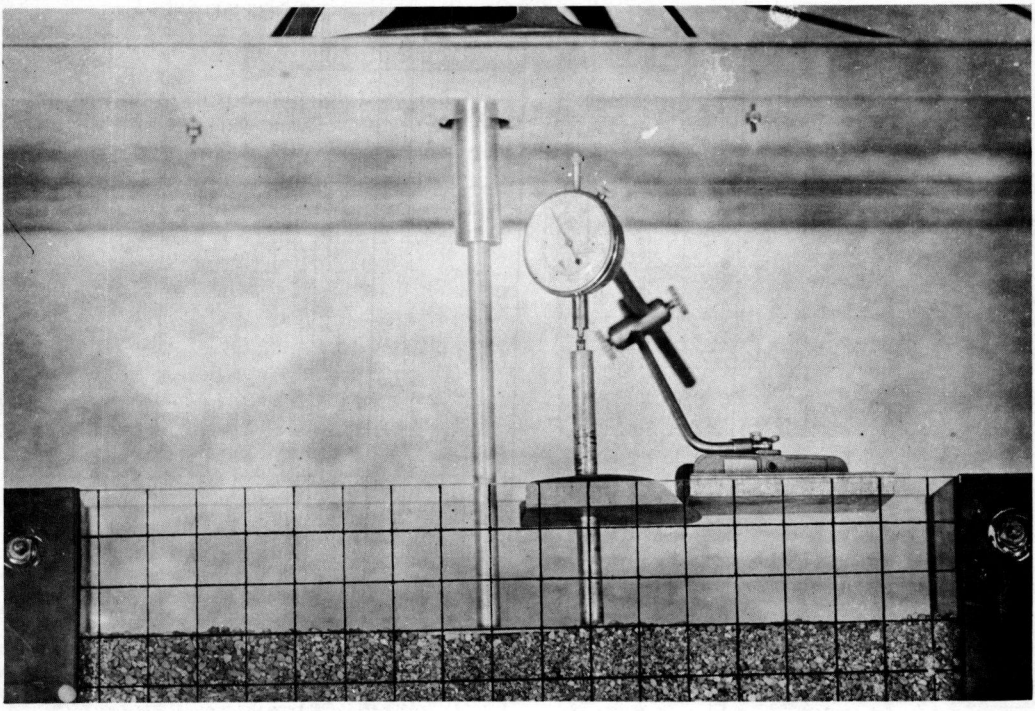

Figure 5. Apparatus for critical frequency.

appreciable settlements (for example, 18 to 30 cps), tests were carried out with all three tampers.

Evaluation of In-Place Density

Each tamper was operated at the critical frequency of 25 cps, during 5 min. Before starting each experiment and at the end of each minute photographs were taken to record the change in the thickness of layers. Examples are shown in Figures 6 through 11.

The reduction in the thickness of the layers is inversely proportional to the increase of the density of sand. Thus the change of the thickness of layers is a measure of their in-place density. On tracing each photograph, the in-place density at every point was calculated and lines drawn connecting equal densities, as shown in Figures 12 through 16.

Evaluation of the Tamping Force

To evaluate the load on the soil from the tamper a proving ring was placed under the tamping rod, as shown in Figure 17. When the tamping rod was vibrated at 25 cps a force of 0.375 lb resulted. Thus the pressures exerted by the 3-, 4-, and 5-in. tampers were 0.045, 0.034, and 0.027 psi respectively.

STANDARD FOR FIELD COMPACTION

Knowing that Proctor curves for sands are erratic and often not sharply defined as to maximum density, relative density was adopted as a standard for the study. The minimum density of the sand was found to be 96.2 pcf by following the funnel method with no circular motion and no free fall, as suggested by D'Appolonia (4). The maximum laboratory density was obtained by the concrete flow table surcharge method of D'Appolonia which resulted in a maximum density of 117 pcf.

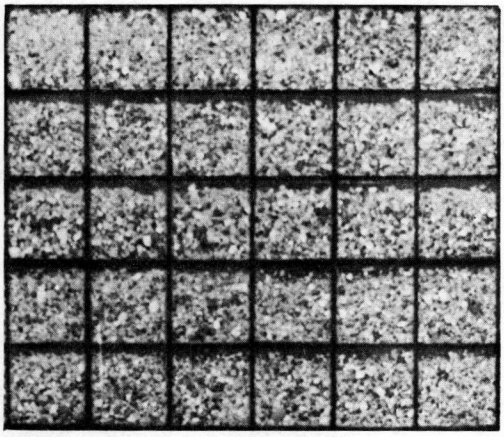

Figure 6. Settlement of 5-in. tamper at 0 min.

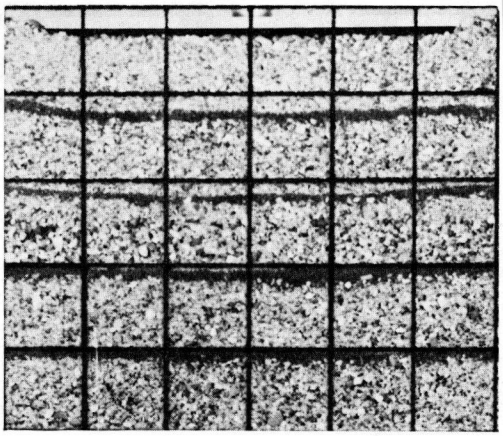

Figure 7. Settlement of 5-in. tamper at 1 min.

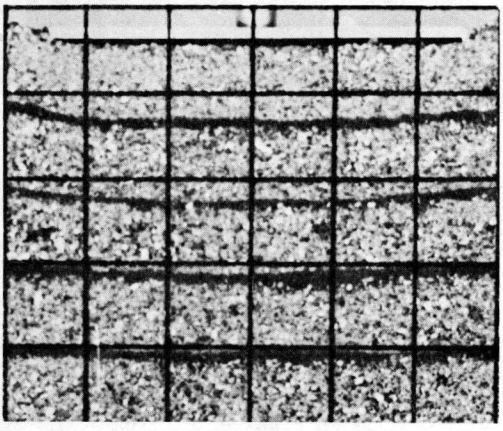

Figure 8. Settlement of 5-in. tamper at 2 min.

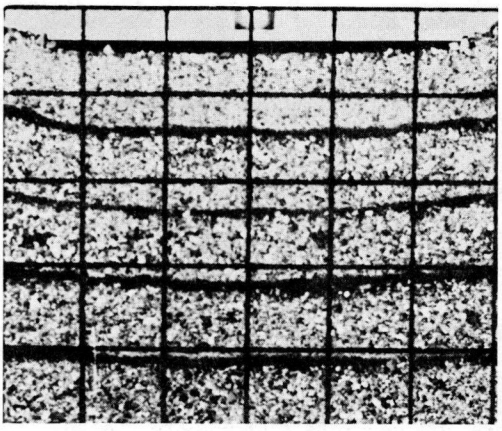

Figure 9. Settlement of 5-in. tamper at 3 min.

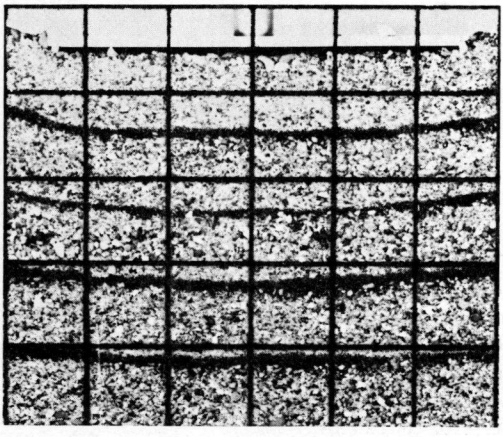

Figure 10. Settlement of 5-in. tamper at 4 min.

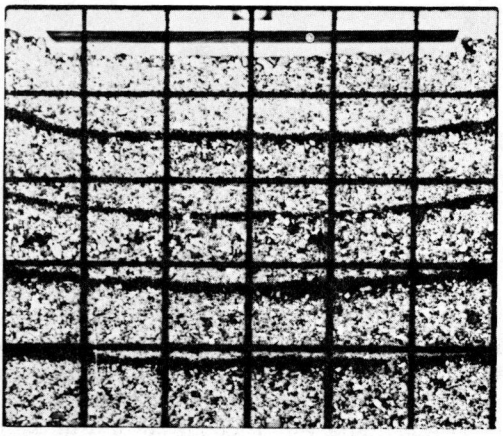

Figure 11. Settlement of 5-in. tamper at 5 min.

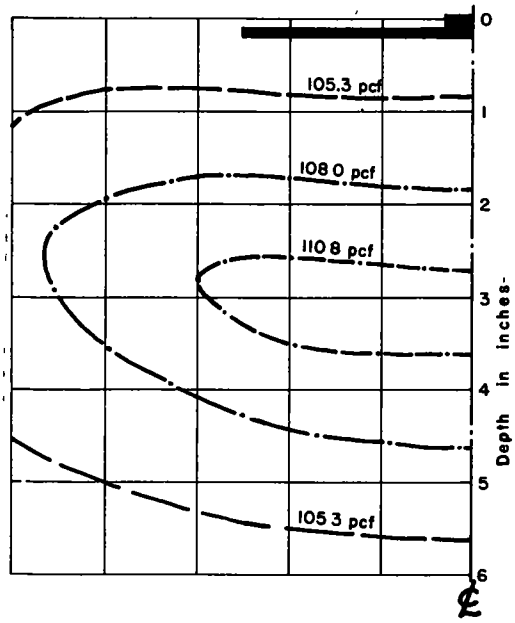

Figure 12. Curves of equal density for 5-in. tamper at 1 min.

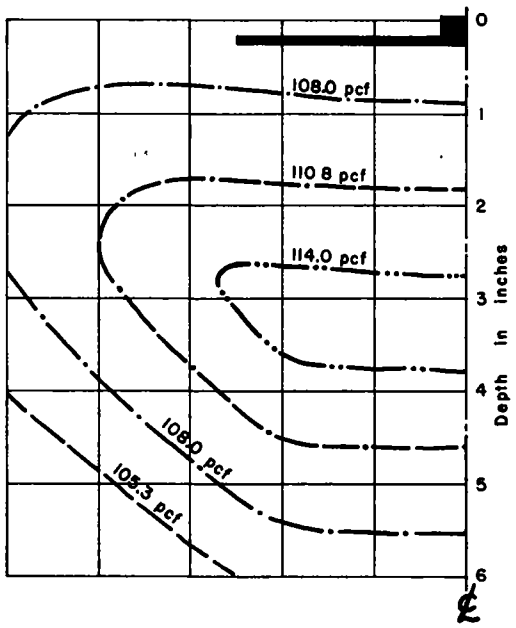

Figure 13. Curves of equal density for 5-in. tamper at 2 min.

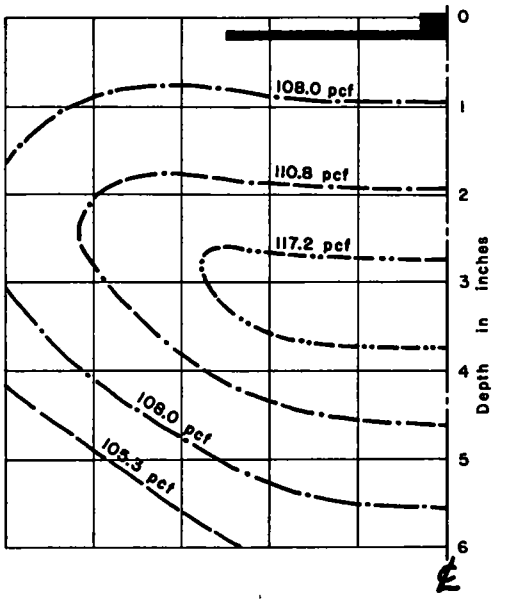

Figure 14. Curves of equal density for 5-in. tamper at 3 min.

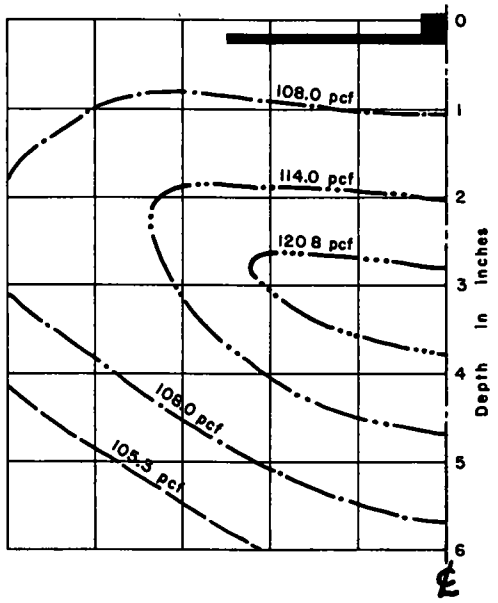

Figure 15. Curves of equal density for 5-in. tamper at 4 min.

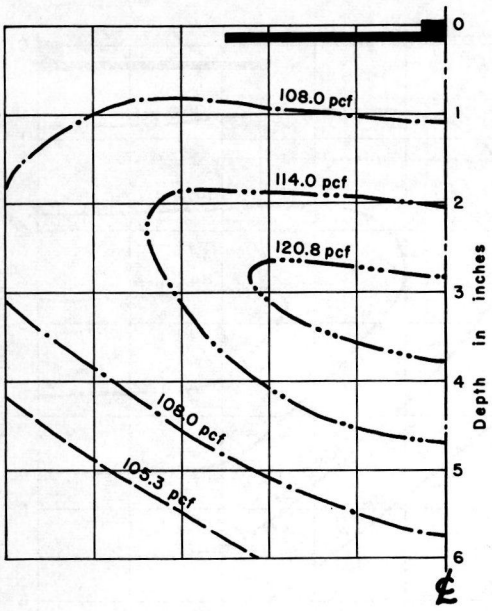

Figure 16. Curves of equal density for 5-in. tamper at 5 min.

RESULTS

The effect on sand settlement and thus density of varying the vibration frequency is shown in Figure 18 and 19. The greatest increase of density was obtained at 25 cps though vibrations above 800 cps did not increase the density at all.

The effect of duration of the vibrations is shown in Figure 20 where it can be seen that 100 percent relative density is reached in 5 or 6 min when the critical frequency of 25 cps is used. The maximum density reached was 120.8 pcf which is larger than the 117 pcf reached by the D'Appolonia method and, therefore, was adopted as the maximum for computing relative density. Also, the area referred to is the region of greatest compaction and not the over-all space beneath the vibrating plates.

Figure 21 shows that this region of greatest compaction moves down from the vibrating plate as the duration increases up to 5 or 6 min but as the plates get larger the ratio of the depth of maximum compaction to the plate width reduces.

The change with time in the depth above which there is 45 or more percent relative

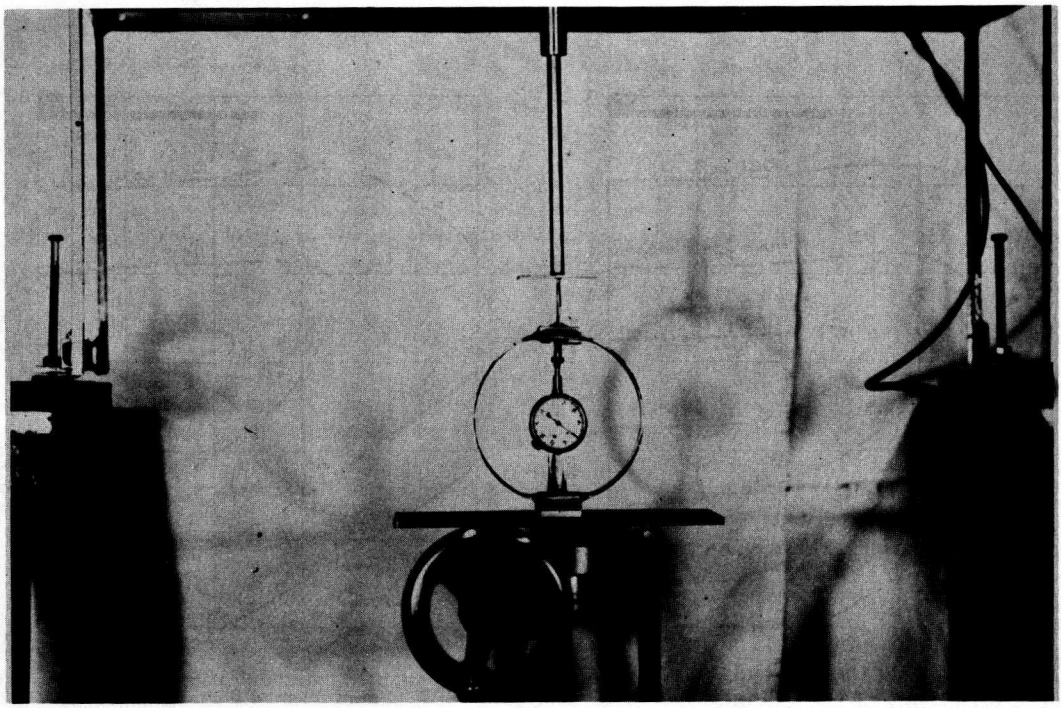

Figure 17. Apparatus for evaluating tamping force.

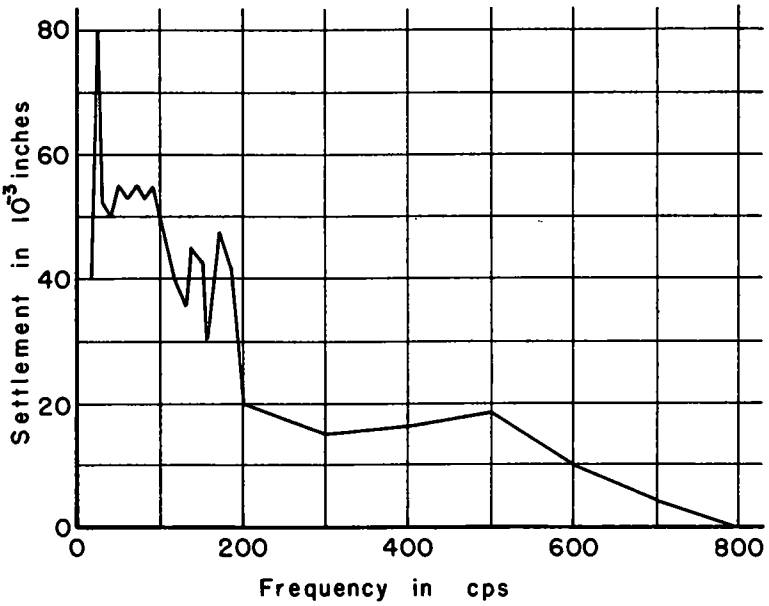

Figure 18. Settlement vs frequency for 3-in. tamper.

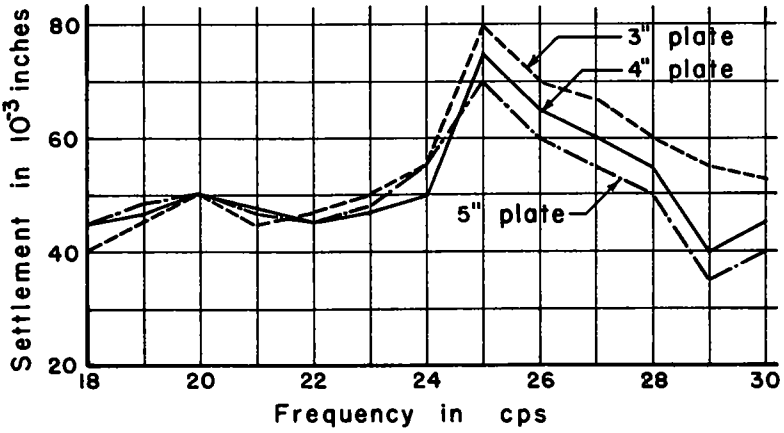

Figure 19. Settlement vs frequency.

density is shown in Figure 22. Here again the ratio of depth of compaction to plate width reduces as the plate size increases.

The change with time in the width within which there is 45 or more percent relative density is shown in Figure 23. Here again the ratio of compacted area to plate width decreases as the plate width increases.

CONCLUSIONS

The results obtained indicate that 25 cps is the most efficient vibration frequency for compacting the dry sand used in the investigation over the tamper-size range used. The efficiency of higher and lower frequencies drops sharply from this optimum indicating that vibrations should not be applied at random frequencies but closely controlled

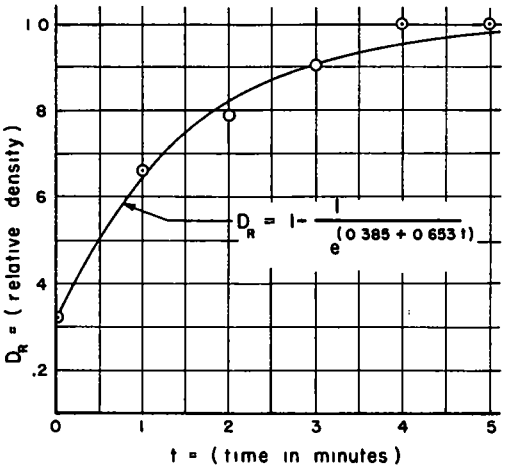

Figure 20. Maximum compaction vs time.

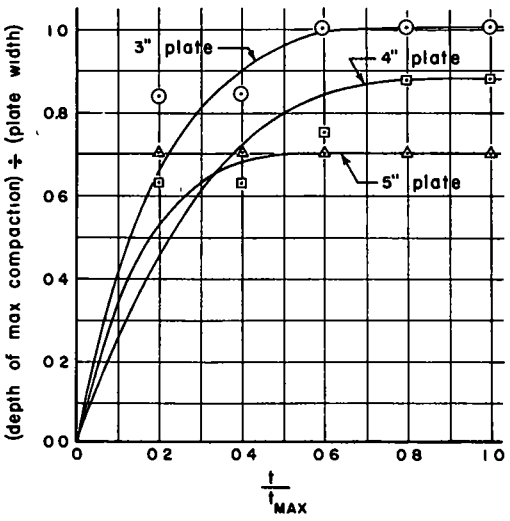

Figure 21. Depth of maximum compaction vs time.

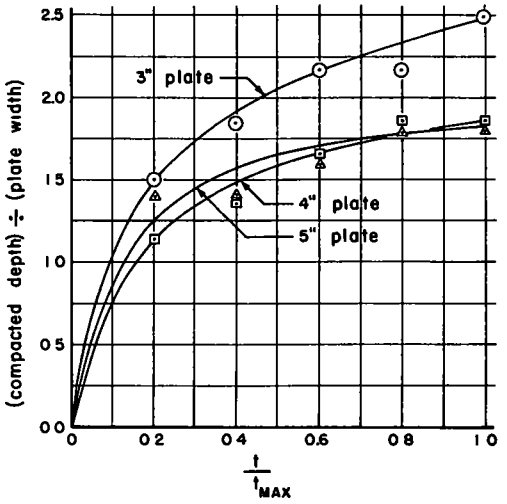

Figure 22. Compacted depth along centerline (D_R = 0.45 or greater) vs time.

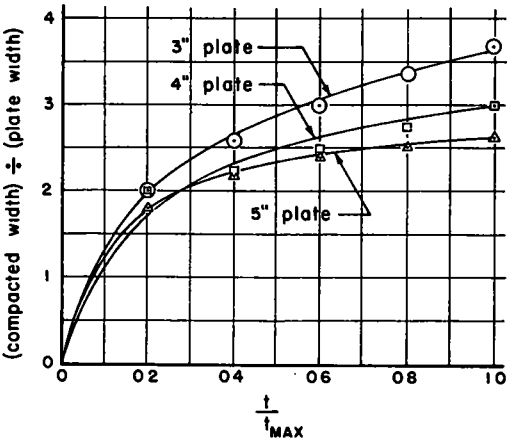

Figure 23. Compacted width at surface (D_R = 0.45 or greater) vs time.

for best results. Comparison of these results with those of other investigators indicates that the optimum frequency may have to be determined for each soil. Even at optimum frequency the vibrations must be applied for an appreciable length of time to obtain reasonable densities.

Maximum compaction is not attained immediately below the tamper but at some distance below the vibrating plate. The ratio of this distance to the plate width decreased as the plate width became larger but not in a straight line variation. There is some evidence that this ratio varied with the tamping force because the tamping force also decreased as the plate size increased.

The following conclusions may be derived from the experiments:

1. Compaction of dry sand by vibration is controlled by the frequency of vibration and is the greatest at the critical frequency.

2. The critical frequency is the one that gives the greatest settlement of surcharge load. For the sand used, the critical frequency was 25 cps.
3. Maximum compaction is not obtained immediately below the tamper but at a certain depth below the surface.
4. No compaction was obtained at supersonic frequencies.
5. The degree of compaction is a function of time and is represented by the equation:

$$D_r = 1 - \frac{1}{e^{(0.358 + 0.653t)}}$$

6. Almost 100 percent compaction is obtained at the end of 6 min at the point of maximum compaction.
7. Surcharge is effective in transmitting the maximum compaction to lower depths.
8. The maximum depth and maximum width to which compaction is effective is an exponential function of tamper dimensions.
9. In evaluating the relative density the minimum laboratory density can be determined by using D'Appolonia's funnel method with no circular motion and no free fall, and the maximum laboratory density can be obtained by vibratory equipment used in this experiment run at critical frequency.

FUTURE SUGGESTED RESEARCH

1. Laboratory maximum density might be determined by using a circular tamper of about a 4-in. diameter with the vibrator used in this experiment. The sand could be continued in a plastic cylinder about 4 in. high with a collar like a Proctor mold. The sand could be placed in four layers. The first layer should be 3 in. thick and the other three layers should be 1 in. thick. Each layer could be compacted at critical frequency for 6 min. The collar could be removed and the excess sand trimmed off as in the Proctor test. The first layer is to permit room for the maximum compaction which would occur in the third inch below the surface with a 4-in. tamper. As the other layers are added, the point of maximum density would move up and the procedure should result in 4 in. of maximum density material.
2. Field compaction by vibrotampers should be run at critical frequencies which could be estimated in situ or determined in the laboratory for each soil.
3. The experiment on dry sand should be repeated with more variety of tamper dimensions to permit correlating the depth of maximum compaction with tamper dimensions.
4. The effect of moisture on the compaction of sand by vibration should be investigated.

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Correlation of Compaction and Classification Test Data

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The results of two correlation studies are reported for a large number of soils from many parts of the United States. In the first study, compaction data are correlated with plastic limit and liquid limit. In the second, compaction data are correlated with different combinations of plastic limit, liquid limit, plasticity index, and several measures of gradation.

• AMONG THE SOIL TESTS required for controlling the quality of highway construction, the compaction test is one of the most important and one of the most time consuming. There is a need for (a) shortening the time required to perform laboratory compaction tests, and (b) developing interrelationships between compaction test data and other laboratory test data to increase the basic knowledge of compaction. The purpose of this paper is to report the results of two studies conducted by the Bureau of Public Roads to accomplish these two objectives.

PUBLISHED CORRELATIONS

In 1938, Woods and Litehiser (1) showed the general interrelation of plastic limit, plasticity index, liquid limit, optimum moisture content, and maximum dry density for 1367 Ohio soils. They reported that increases in the plastic properties of the soils were accompanied by increases in optimum moisture content and decreases in maximum dry density.

In a more recent report, Jumikis (2) presented a chart (Fig. 1) relating optimum moisture content obtained from "standard soil compaction tests" with liquid limit and plasticity index for various New Jersey glacial soils.

Rowan and Graham (3) presented two formulas for estimating the maximum dry density and the optimum moisture content as determined in the Proctor test. (The details of their compaction test were not given. In the original Proctor test (4), 25 firm 12-in. strokes of a 5.5-lb tamper were used on each of three layers of soil in a mold about 4 in. in diameter and 5 in. in height.) These formulas as follows:

$$\text{Calculated density (pcf)} = \frac{D}{1 + \frac{D - C}{62.5S}} \quad (1)$$

$$\text{Calculated optimum moisture (percent)} = SL\left(\frac{B}{A}\right) \quad (2)$$

in which

$D = \frac{CA}{B}$;

C = 62.5 x shrinkage ratio, pcf;

A = percentage passing No. 4 sieve;

B = percentage passing No. 40 sieve;

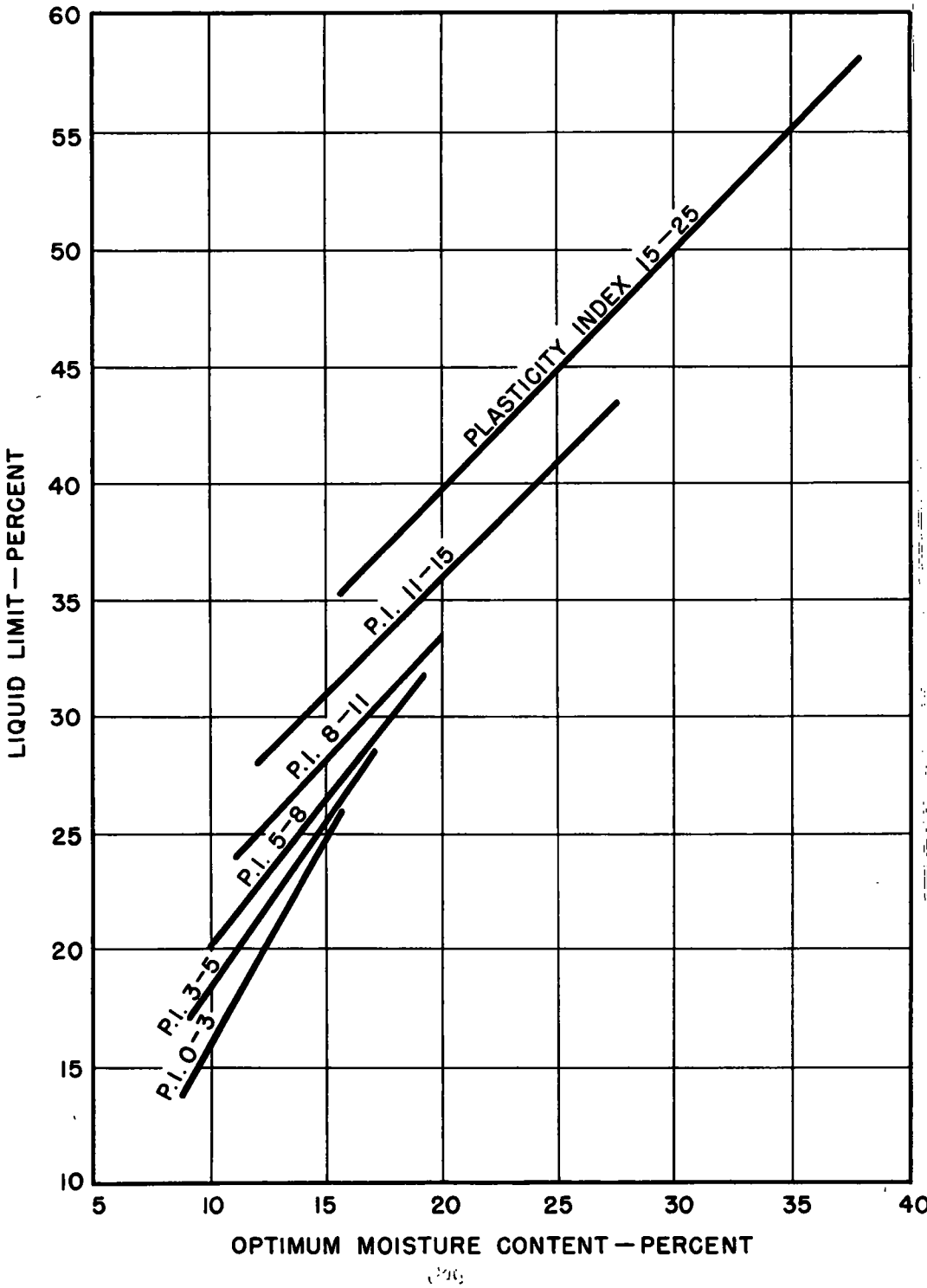

Figure 1. Optimum moisture content and liquid limit related to plasticity index of various glacial materials, after Jumikis (2).

S = specific gravity; and

SL = shrinkage limit, percent.

In comparing the predicted values with test values for 10 soils, Rowan and Graham found that the greatest deviation (difference between the predicted value and the test value) in maximum dry density was about 5 percent and that the predicted optimum moisture content was slightly high, ranging from 1 to 5 percentage points above the test values. They suggested that all calculated (predicted) optimum moisture contents be corrected by subtracting 3 percentage points.

Davidson and Gardiner (5) used the Rowan-Graham formulas in an analysis of test data for 210 soils from 11 States. They found wide deviations between the predicted and test values obtained from the standard Proctor control tests on highly plastic soils. They determined that the size of the deviation was related to the plasticity index of the soil and revised the Rowan-Graham formulas to fit these data more closely. The revised formulas are as follows:

$$\text{Calculated density (pcf)} = \frac{6250\ K_1}{SL\left(\frac{B}{A} - 1\right) + \frac{100}{R}} \quad (3)$$

$$\text{Calculated optimum moisture (percent)} = SL\left(\frac{B}{A}\right) + K_2 \quad (4)$$

in which

$$K_1 = \frac{312 - 2(PI)}{300};$$

PI = plasticity index;

R = shrinkage ratio; and

$$K_2 = \frac{PI}{3} - 4.$$

Eq. 3 does not include the specific gravity term S which appears in Eq. 1. The S-values used by Rowan and Graham were calculated from shrinkage test data, whereas Davidson and Gardiner have substituted the shrinkage data directly into the formula.

Turnbull (6) showed that the optimum moisture content is closely related to the gradation of the soil. For a numerical measure of gradation, he used the area above the grain-size distribution curve and named it the classification area (7). The solid curved line in Figure 2 shows the relationship of classification area to optimum moisture content for 101 soils tested by a compaction test method very similar to AASHO T99, Method A. (The compaction effort is 15 percent greater.) This simple curvilinear relationship (Fig. 2) fits the test values of optimum moisture content very closely; 72 percent of the predicted values are within 1.0 percentage point of the test values. Additional tests made later by Turnbull on coarse-grained soils, with high classification areas, indicate that the curve in Figure 2 should be corrected to follow the dotted line (8).

To simplify the determination of the classification area, Turnbull subdivided the grain-size distribution chart by equally spaced ordinates. Figure 3 shows an adaptation of Turnbull's chart. The original chart by Turnbull extends to the right five more units to include particle sizes up to 6 in. The extra five units were not needed in the Public Roads study because 4.7 mm (about $^3/_{16}$ in.) was the maximum size particle used in the compaction test.

To determine the classification area from the grain-size distribution curve of a given soil material, ½ of the length (in percent) of ordinate 0 above the curve is added to the sum of the lengths of ordinates 1 through 19 above the curve, and that sum is multiplied by 0.00301. For example, the gradation of a sample of Cecil coarse sandy loam has been plotted in Figure 3. The length of the ordinates (to be added) above the

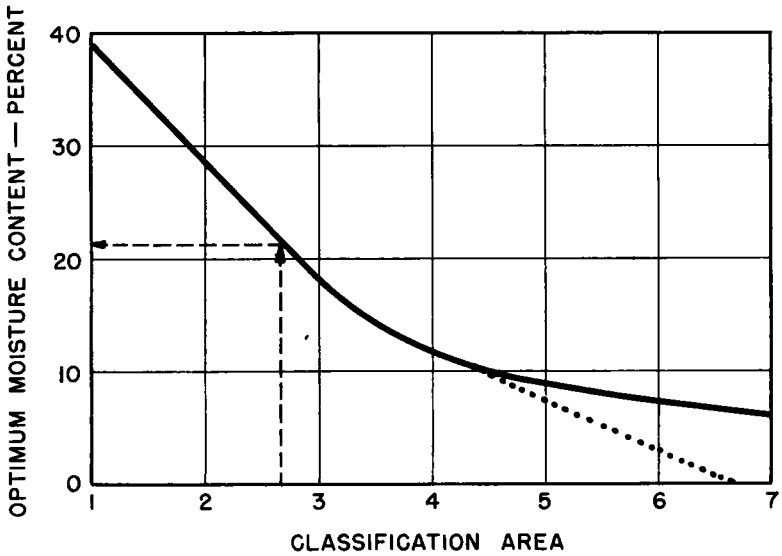

Figure 2. Optimum moisture content vs classification area, based on test data from 101 soils (6). Dotted line shows position of corrected curve, based on additional tests (8) of coarse material.

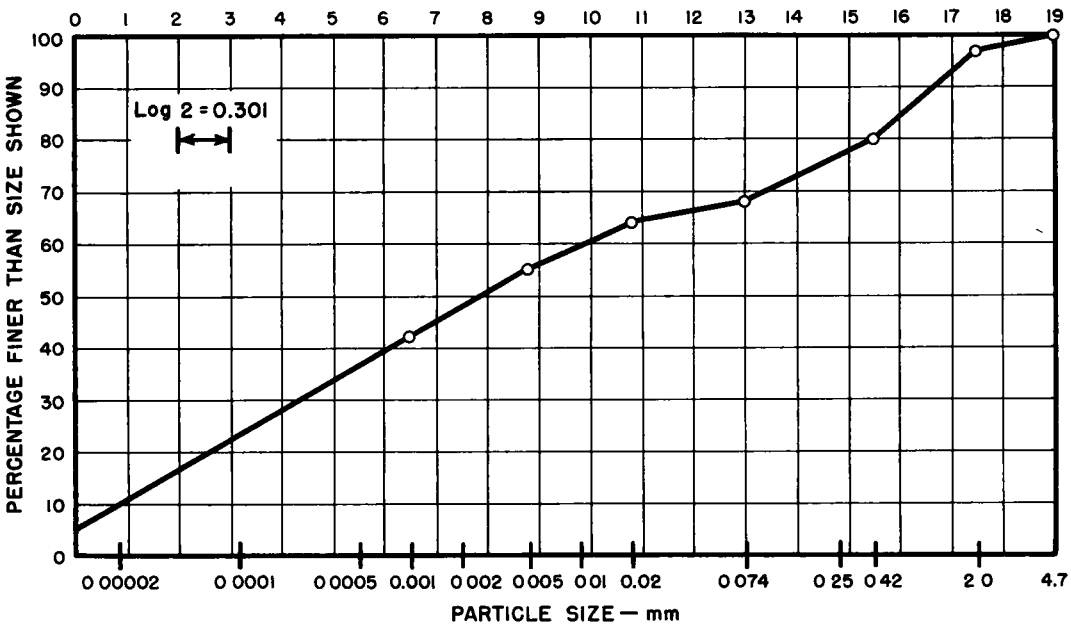

Figure 3. Grain-size distribution of Cecil coarse sandy loam on a chart for determining Turnbull classification area, adapted from Turnbull (7).

curve are $\frac{95}{2}$, 89, 84, 78, 72, 67, 61, 55, 50, 44, 40, 36, 34, 32, 27, 23, 16, 8, 2, and 0. The sum of these, 866, when multiplied by 0.00301 yields a classification area of 2.68. By locating the point corresponding to this area in Figure 2, a value of 22 percent is obtained for the optimum moisture content.

FIRST PUBLIC ROADS STUDY

The first attempt by the Bureau of Public Roads (1958) to correlate optimum moisture content and maximum dry density with classification data was based on test data of 972 soil samples from 31 states. The compaction test used in this study (as well as in the second study) was performed in accordance with AASHO Designation T99-49, which is the same as the current AASHO Designation T99-57, Method A. In this method, the soil fraction passing the No. 4 sieve is compacted in three layers in a 4-in. diameter mold by dropping a 5.5-lb rammer from a height of 12 in. 25 times per layer.

Correlations were made by plotting the test data on rectangular coordinates. The chart (Fig. 4) developed by Yemington (9) in this study, correlates optimum moisture content and maximum dry density with plastic limit and liquid limit.

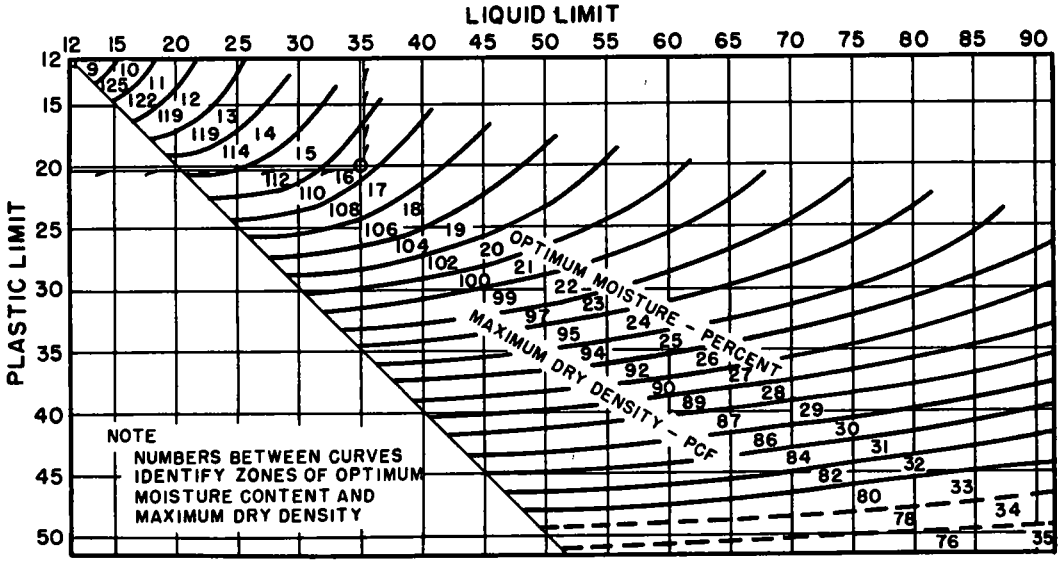

Figure 4. Relation of average maximum dry density and optimum moisture content (AASHO T99-49) to plastic limit and liquid limit.

To evaluate the chart, it was used to estimate values of optimum moisture content for 510 additional soil samples from a number of states. These estimated values were compared to test values, with results as given in Table 1. The comparison shows that 81 percent of the predicted optimum moisture contents are within 2 percentage points of the test values. The correlation is best for eastern soils. Of the predicted optimum moisture contents for the 222 samples from east of the Mississippi River, 94 percent are within 2 percentage points of the test values. Soils showing least correlation are from "non-soil" areas west of the Mississippi River.

To evaluate estimates of maximum dry density from the chart (Fig. 4), a study was made of test data from 532 samples, which included the 510 samples referred to

TABLE 1

SUMMARY OF DEVIATIONS OF OPTIMUM MOISTURE CONTENTS[1]

State	Predominant Soil Type (Origin)[2]	No. of Samples for Which Estimated Optimum Moisture Content Is Less Than Test Value by											No. of Samples for Which Estimated Optimum Moisture Content Exceeds Test Value by						
		-11	-10	-9	-8	-7	-6	-5	-4	-3	-2	-1	0	+1	+2	+3	+4	+5	+6
Ala.	Residual	-	-	-	-	-	-	-	-	-	-	3	16	12	1	-	-	-	-
Ariz.	Residual	-	-	-	-	-	1	1	2	1	-	-	2	-	-	-	-	1	-
Ark.	Recent alluvium	-	-	-	-	-	-	-	1	1	2	4	10	8	2	-	-	-	-
Conn.	Glacial	-	-	-	-	-	-	-	1	1	1	4	7	2	2	1	-	-	-
Fla.	Coastal Plain sand	-	-	-	-	-	-	-	-	1	1	1	3	4	1	-	-	-	-
Idaho	Non-soil	1	1	2	-	-	4	5	4	4	6	11	20	9	2	-	3	-	1
Ill.	Loessial	-	-	-	-	-	-	-	-	-	1	10	12	5	-	-	-	-	-
Ky.	Residual	-	-	-	-	-	-	-	-	1	-	6	18	4	1	-	-	-	-
Md.	Residual	-	-	-	-	-	-	-	-	-	2	3	6	7	3	1	2	-	-
Minn.	Glacial	-	-	-	-	-	-	-	2	4	4	4	3	2	1	-	-	-	-
Neb.	Outwash	-	-	-	-	-	-	1	1	7	7	7	15	7	4	1	1	-	-
	Loessial	-	-	-	-	-	-	-	-	2	4	13	8	-	1	-	-	-	-
Nev.	Non-soil	-	-	1	1	2	1	1	2	1	-	-	-	-	-	-	-	-	-
N.M.	Residual	-	-	-	-	-	-	-	-	-	1	2	4	1	2	-	-	-	-
N.C.	Residual	-	-	-	-	-	-	-	-	-	1	3	5	5	1	2	-	1	-
N.D.	Glacial	-	-	-	-	-	-	1	2	2	2	2	4	4	2	-	-	-	-
Ore.	Non-soil	-	-	-	-	1	2	2	1	1	1	2	7	-	3	-	-	-	-
Tenn.	Residual	-	-	-	-	-	-	1	3	-	10	11	7	6	-	-	-	1	-
Texas	Coastal Plain clay	-	-	-	-	-	-	-	-	-	-	2	7	1	2	-	1	-	-
	Residual	-	-	-	-	-	-	-	1	1	2	-	-	1	2	-	-	-	-
Vt.	Glacial	-	-	-	-	-	-	1	-	1	-	3	4	7	-	-	-	-	-
Ohio	Lacustrine	-	-	-	-	-	-	-	1	-	1	1	2	2	-	-	-	-	-
	Total	1	1	3	1	3	8	13	21	28	46	92	160	87	30	5	7	3	1

[1] Estimated by the PL and LL chart (Fig. 4) from those determined by test, AASHO T99-57, Method A.

[2] After Belcher et al (10).

previously. Sixty-three percent of the values were within 4 pcf of the corresponding test values.

Another appraisal of the chart was made with test data for soil samples obtained from Alaska and places outside the continental United States. The comparisons, given in Table 2, ranged from reasonably good for soils from Alaska to extremely poor for those from Costa Rica, Panama, and Hawaii. Whereas the data clearly show that wide variations occur between predicted and actual test values for certain soils, the variety of soils studied is too limited to warrant conclusions as to the cause of the variations.

The results of this first study proved to be very useful. The chart, (Fig. 4) has been used for several years as a guide in performing compaction tests, particularly to estimate the amount of water to use for the first moisture-density point. It has also been used by the laboratory supervisor to determine if the optimum moisture content values obtained by technicians were reasonable. The use of the chart is limited, however, in that it does not "fit" a large number of unusual soils. Further analysis was needed to make the correlations applicable to a wider range of soils and to make the estimates more accurate.

TABLE 2

COMPARISONS OF OPTIMUM MOISTURE CONTENTS AND MAXIMUM DRY DENSITIES[1] FOR SOILS FROM ALASKA AND OUTSIDE THE CONTINENTAL UNITED STATES

Soil		Optimum Moisture Content			Maximum Dry Density		
Location	Kind	Test Value (%)	Estimated (%)	Deviation	Test Value (pcf)	Estimated (pcf)	Deviation
Alajuela, Costa Rica	Lateritic	41	22	-19	77	99	+22
Las Lomas, Panama	Lateritic	29	19	-10	88	104	+16
Liberia	Lateritic	20	24	+ 4	107	95	-12
		12	14	+ 2	126	114	-12
		15	17	+ 2	117	108	- 9
Addis Ababa, Ethiopia	Black Cotton	39	29	-10	80	87	+ 7
Oahu, Hawaii	Kaolinitic	33	22	-11	85	99	+14
		36	23	-13	80	97	+17
	Montmorillonitic	37	30	- 7	79	86	+ 7
		35	27	- 8	84	90	+ 6
Kodiak, Alaska		20	21	+ 1	103	100	- 3
		24	23	- 1	95	97	+ 2
		33	30	- 3	80	86	+ 6
Kenai-Kasilof, Alaska		10	10	0	131	125	- 6
		14	14	0	118	114	- 4
		13	12	- 1	124	119	- 5

[1]Estimated by PL and LL chart (Fig. 4) with those determined by test, AASHO T99-57, Method A. Data from Public Roads laboratory, except for Hawaiian soils which are from Kawana and Homes (11).

SECOND PUBLIC ROADS STUDY

To improve the methods for predicting optimum moisture content and maximum dry density, a second study was made in 1961 using multiple linear regression analysis. This method of analysis permits all of several variables to be used jointly for estimating the desired value.

Selection of Samples and Variables

Soil test data were selected from the files on the basis of the geographical and geological origin of the soil samples. Data were selected to represent a broad coverage of soils within the continental United States. Initially, 946 samples were selected, many of which were the same as used in the first study. This number was reduced to 600 by the use of a set of random numbers. The nonplastic soils were eliminated, after preliminary analyses of the test data showed different interrelationships from those of the plastic soils. The analyses were made on the remaining 527 samples of plastic soil. The general types (origins) of soils represented, their sampling locations, and the number of samples from each location are shown in Figure 5.

The independent variables used in the analyses included plastic limit, liquid limit, plasticity index, and several measures of gradation. Specific gravity was also considered but was not used because of insufficient data. Standard AASHO tests were used in determining the plasticity and gradation test data; gradation was determined on the fraction passing the No. 4 sieve because that was the fraction used in the compaction test, AASHO T99-57, Method A. Gradation, as represented by percentages passing specific sieves, could not be used as an independent variable because the regression type of analysis requires that each variable be expressible or measurable

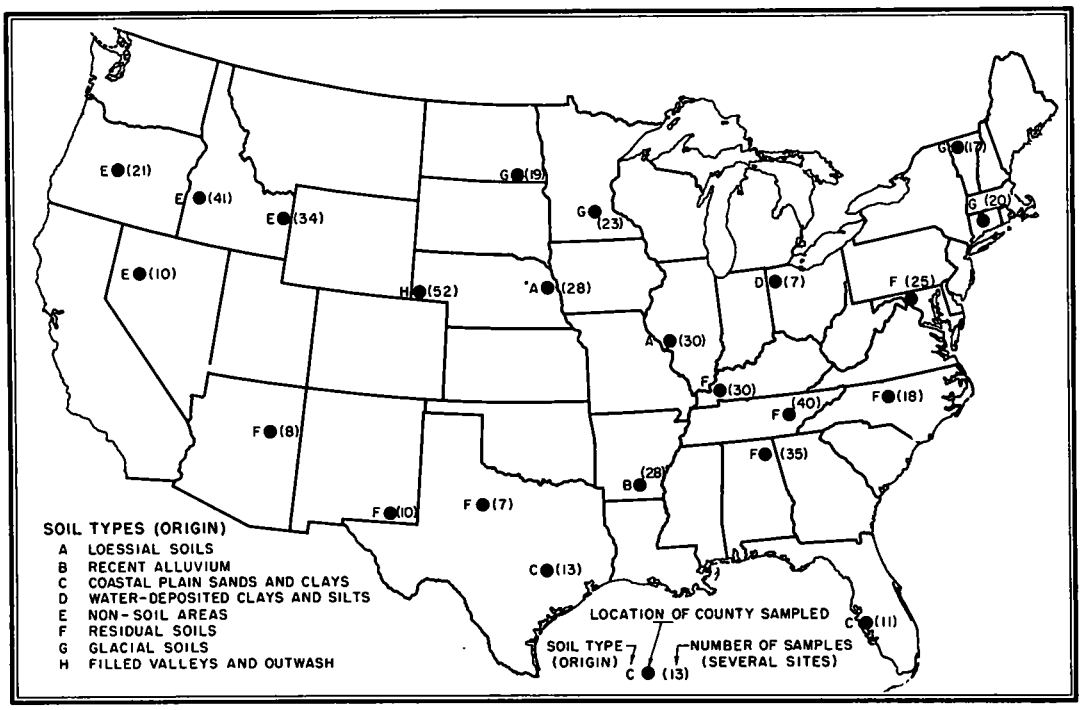

Figure 5. Primary soil type (origin), location of county sampled and number of samples.

by a single number. To represent characteristics of gradation, several measures were tried.

Burmister (2) reported, "The significant characteristics of grain-size distribution are fineness, range of grain size, and type of grading." All of these characteristics were evaluated in the second Public Roads study.

Three measures were devised for fineness:

1. Percentage of particles finer than the 0.001-mm size, designated as 0.001 fraction.
2. Fineness average, FA. This is determined by taking one-sixth of the total of the percentages, by weight, finer than the following sizes in millimeters: 2.0 (No. 10), 0.42 (No. 40), 0.074 (No. 200), 0.020, 0.005, and 0.001.
3. The average particle size, D'_{60}. This is obtained at the midpoint of a straight line (Fig. 6) drawn to approximate the major portion of the grain-size distribution curve. The dashed line in the figure is an example of this straight line; it approximates the grain-size distribution curve from D_{10} to D_{90}.

The percentage finer than the 0.001-mm size was used in the first two measures above mainly because it is the finest fraction normally measured in the Public Roads soils laboratory.

The range of grain or particle sizes, designated herein as "range," is defined as the number of log cycles traversed by the straight line approximating the D_{10} to D_{90} portion of the curve.

The type of grain-size distribution curve is designated "shape." The five shapes considered are shown in Figure 6.

Examination of Simple Relationships

To examine the simple relationships of each dependent variable (optimum moisture content and maximum dry density) with each independent variable, plots were made on

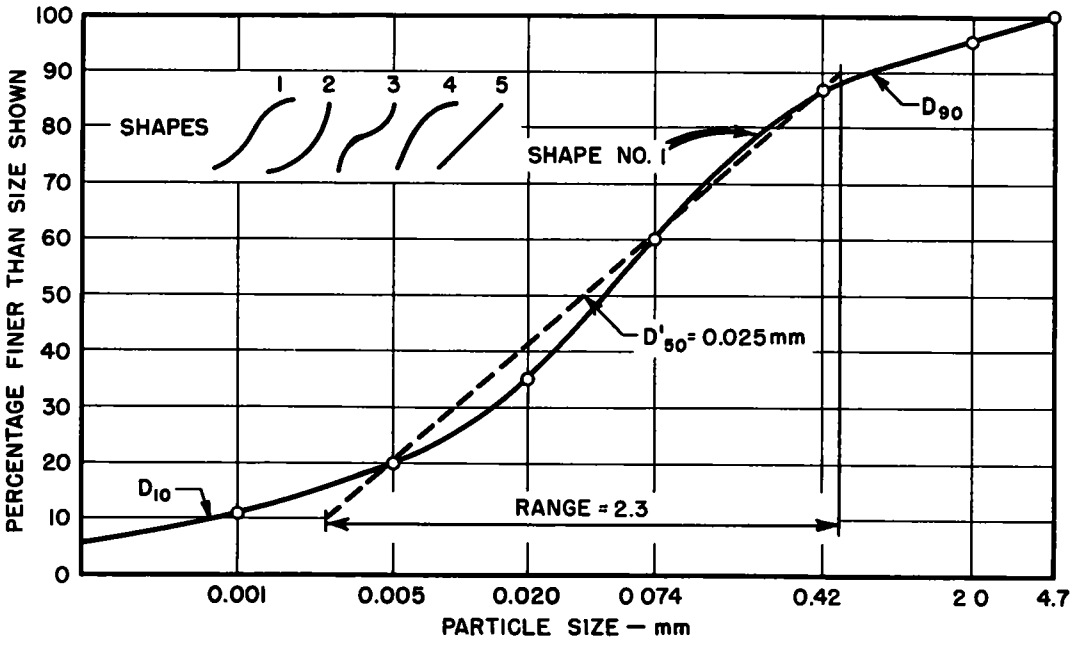

Figure 6. Typical grain-size distribution curve showing shape, D'_{50}, and range.

rectangular coordinates to arithmetic scale. Separate plots were made for each "shape." An example — optimum moisture content vs liquid limit for shape 2 — is given in Figure 7. The results indicated good correlations of optimum moisture content with liquid limit and with plastic limit, and good correlations of maximum dry density with optimum moisture content and with plastic limit. A summary of findings is given in Table 3. In each case where a definite linear or curvilinear relation was developed on arithmetic scale, that line approximating the data was replotted to logarithmic scale. The type of relationship resulting — linear or curvilinear — is also shown in the table.

An examination of the plotted data for each "shape" indicated that separating the data on the basis of "shape" was of little or no value; hence, the data for all the samples were subsequently analyzed together regardless of shape.

Regression Analysis by Electronic Computer

The multiple linear regression analyses were performed on an IBM 650 electronic computer using a computer program (Multiple Regression Analysis, written by Arthur Cohen, I.B.M. 650 program library, file number 6.0.001) supplied by the machine manufacturer. The multiple regression analysis is a method for obtaining a formula for estimating one variable by means of several other variables. The analysis provides the linear equation that best fits the data.

The results of the multiple linear regression analyses are summarized in Table 4. The variables and the standard error of estimate are given for each analysis. The formulas that were developed are listed in Table 5. The standard error of estimate is a measurement of deviation or degree of scatter of the points (test values) around the regression line. If the normal distribution of error holds, 67 percent of the test values will be within one standard error of the predicted value, 95 percent will be within 2 standard errors. The unit of measure is the same as that of the predicted variable.

In the first of five regression analyses to determine equations for predicting optimum moisture content, a relationship was sought using all six independent variables. The analysis was made without adjustments for the curvilinear relationships of D'_{50} and FA (Table 4). The resulting standard error of estimate was ±2.00 percent moisture.

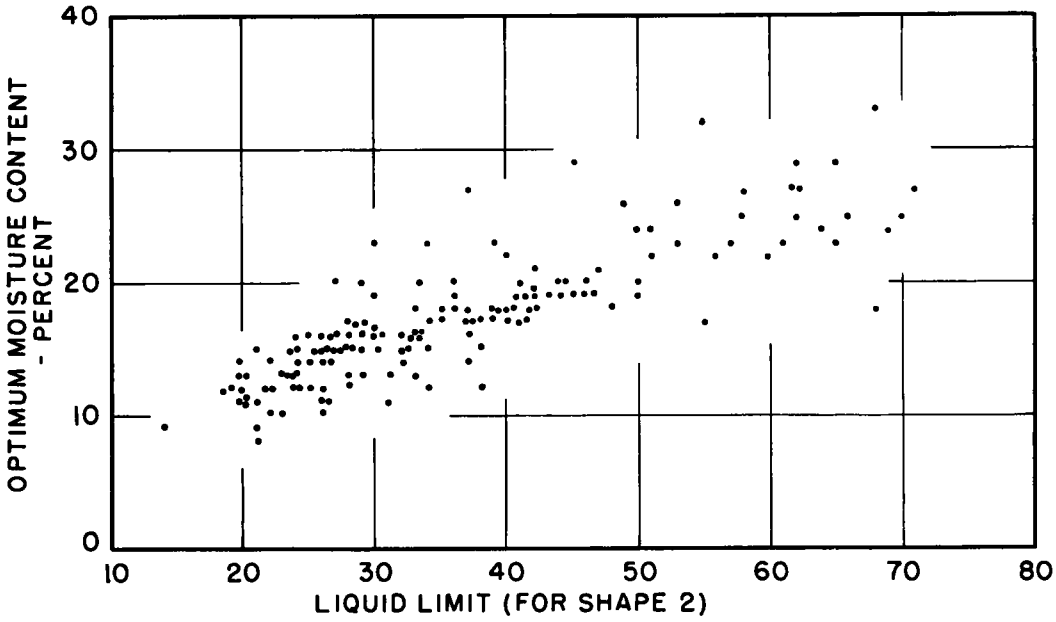

Figure 7. Preliminary plot of optimum moisture content vs liquid limit.

TABLE 3

CORRELATION BETWEEN VARIABLES[1] AS DETERMINED BY INSPECTION OF DATA PLOTTED ON RECTANGULAR COORDINATES

Correlation		Rating of Correlation[2]			Type of Relationship on Arithmetic Scale		Type of Relationship on Log-Log Scale	
		Good	Fair	Poor	Linear	Curved	Linear	Curved
Optimum moisture content vs:	LL	x			x		x	
	PL	x			x		x	
	PI		x		x			x
	Range			x				
	D'_{50}		x			x	x	
	FA		x			x		x
	0.001		x		x			x
Maximum dry density vs:	OMC	x				x		x
	LL		x		x			x
	PL	x			x			x
	PI		x					
	Range			x				
	D'_{50}		x					
	FA		x			x		x
	0.001		x					

[1]LL = liquid limit; PL = plastic limit; PI = plasticity index; Range = number of log cycles traversed by straight line approximating D_{10} to D_{90} portion of grain-size distribution curve; D'_{50} = average particle size, determined at midpoint of straight line referred to in "Range" definition; FA = fineness average, equal to 1/6 sum of percentages finer than mm sizes 2.0, 0.42, 0.074, 0.020, 0.005, and 0.001; 0.001 = percentage of particles finer than 0.001 mm; OMC = optimum moisture content.

[2]Based on degree of scatter of plotted points about line of best fit.

TABLE 4

VARIABLES AND RESULTING STANDARD ERROR OF ESTIMATE FOR EACH REGRESSION ANALYSIS

Analysis No.	Dependent Variable	Independent Variables[1]							Standard Error of Estimate[2]	Remarks
		OMC	LL	PL	PI	D_{50}	FA	0.001		
1	OMC		x	x	x	x	x	x	±2.00	Data used directly
2	OMC		x	x	x	x	x	x	±2.05	Log transformation[3]
3	OMC		x	x	x	x	x	x	±1.98	Log transformation with adjustments[4]
4	OMC			x			x		±2.17	Log transformation with adjustments[4]
5	OMC			x			x		±1.13	Log transformation with adjustments[4,5]
6	MDD		x	x			x		±4.44	Data used directly
7	MDD	x	x	x			x		±2.52	Log transformation with adjustments[4]
8	MDD			x			x		±4.32	Log transformation with adjustments[4]
9	MDD			x			x		±2.98	Log transformation with adjustments[4,5]

[1] Simplified; see Table 5 for exact form of each. Meanings of abbreviations are given in Table 3.
[2] Standard error of estimate by predicting with PL alone is ±2.45; predicting equation is OMC = 0.811 PL - 0.430.
[3] Values of all variables transformed to natural logarithms.
[4] Constants added to some independent variables before logarithmic transformation to make all relationships linear with dependent variable.
[5] Analysis for 40 samples from Loudon Co., Tenn.

In analysis 2, logarithmic transformations were made of all variables in order to make the linear program applicable to curvilinear relationships. The resulting standard error of estimate indicated a slightly poorer correlation than that obtained in analysis 1. It was found that the logarithmic transformation, in addition to straightening out certain curvilinear relationships, had caused some linear relationshps to become curvilinear.

Before analysis 3, the average linear and curvilinear relationships developed in arithmetic plots of the data were plotted to logarithmic scale to determine the effect of logarithmic transformation on all the variables. In cases where the resulting plot was curved, constants were added to the independent variables to obtain straight line relationships. An example of this type of adjustment is shown in Figure 8. The constants were determined by trial and error. The standard error of estimate of ±1.98 percent moisture indicated a slightly better relationship than found in the previous analysis.

Analysis 4 compared optimum moisture content with only two independent variables — plastic limit and fineness average. The number of variables was reduced to simplify the predicting equation. Plastic limit and fineness average were used because the partial correlation coefficients from previous analyses had indicated them to be the best two of the independent variables for predicting optimum moisture content. The reduction of independent variables, however, reduced the accuracy of the predicting formula, the standard error of estimate increasing to ±2.17 percent moisture. The predicting formula developed in this analysis is:

TABLE 5

SUMMARY OF PREDICTING FORMULAS FROM PUBLIC ROADS STUDY 2

Analysis No.	Predicting Formula[1]
1	$OMC = 1.427\ LL - 0.815\ PL - 1.373\ PI - 0.0007\ D'_{50} + 0.062\ FA + 0.035\ (0.001\ \text{fraction}) - 1.312$
2	$\log OMC = 0.158 \log \left(\frac{LL}{10}\right) + 0.647 \log PL + 0.021 \log PI + 0.012 \log \left(\frac{D'_{50}}{100}\right) + 0.354 \log FA + 0.248 \log (0.001\ \text{fraction}) - 0.974$
3	$\log OMC = 1.029 \log LL + 0.045 \log PL + 0.224 \log (PI + 15) - 0.033 \log \left(\frac{D'_{50}}{100}\right) + 0.229 \log (FA + 100) + 0.098 \log (0.001\ \text{fraction} + 40) - 3.401$
4	$\log OMC = 0.784 \log PL + 1.378 \log (FA + 100) - 6.586$
5[2]	$\log OMC = 0.763 \log PL + 1.377 \log (FA + 100) - 6.544$
6	$MDD = 147.525 - 0.020\ LL - 1.195\ PL - 0.198\ FA$
7	$\log MDD = 7.126 - 0.653 \log (OMC + 15) + 0.059 \log LL - 0.120 \log (PL + 20) + 0.014 \log FA$
8	$\log MDD = 7.247 - 0.567 \log (PL + 20) - 0.110 \log FA$
9[2]	$\log MDD = 7.105 - 0.518 \log (PL + 20) - 0.113 \log FA$

[1]OMC = optimum moisture content, AASHO T99-57, Method A; MDD = maximum dry density, AASHO T99-57, Method A; LL = liquid limit; PL = plastic limit; PI = plasticity index; D'_0 = average particle size on straight line approximating grain-size distribution curve; FA = fineness average; 0.001 fraction = percentage of particles finer than 0.001 mm; log = natural logarithm.
[2]For 40 samples from Loudon County, Tenn.

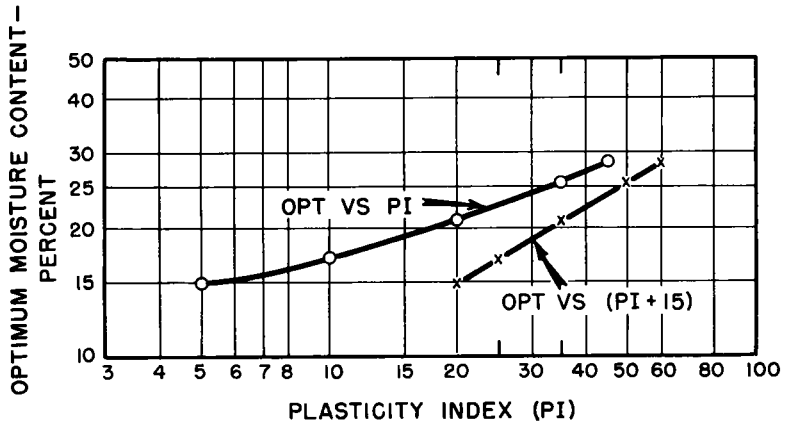

Figure 8. Logarithmic plots of optimum moisture content vs PI and optimum vs (PI + 15), showing effect of adding constant to independent variable.

$$\log OMC = 0.784 \log PL + 1.378 \log (FA + 100) - 6.586 \quad (5)$$

in which

log = natural logarithm;

OMC = optimum moisture content, percent;

PL = plastic limit; and

FA = fineness average.

Figure 9, developed from the above formula, shows optimum moisture content values for the range of plastic limits and fineness average values studied.

The standard errors of estimate shown in Table 4 are based on the numerical differences between the actual and predicted values. These standard errors of estimate may give the erroneous impression that the dependent variable can be predicted with the same accuracy at both high and low values. A more realistic picture of the relationship between predicted values from Eq. 5 and test values is shown in Figure 10. The deviations, in percent moisture, increase as the test values increase.

Analysis 5 was made with data representing 40 samples from Loudon County, Tenn. The test data from a single county were selected to show how closely a regression formula would fit actual test results when the soils were from a relatively small area,

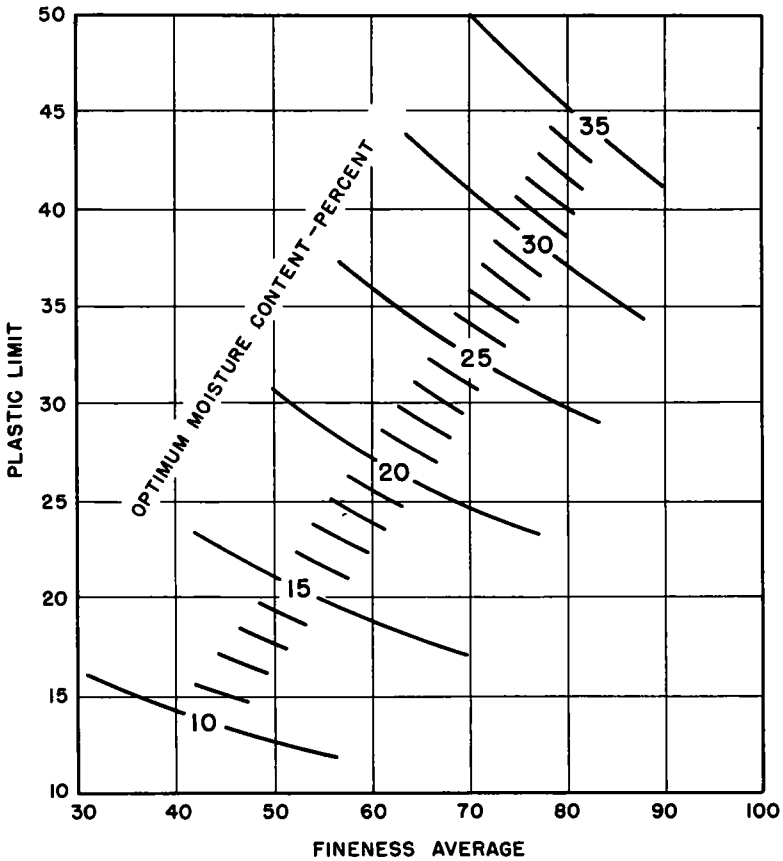

Figure 9. Relation of optimum moisture content (AASHO T99-57, Method A) to plastic limit and fineness average, analysis 4.

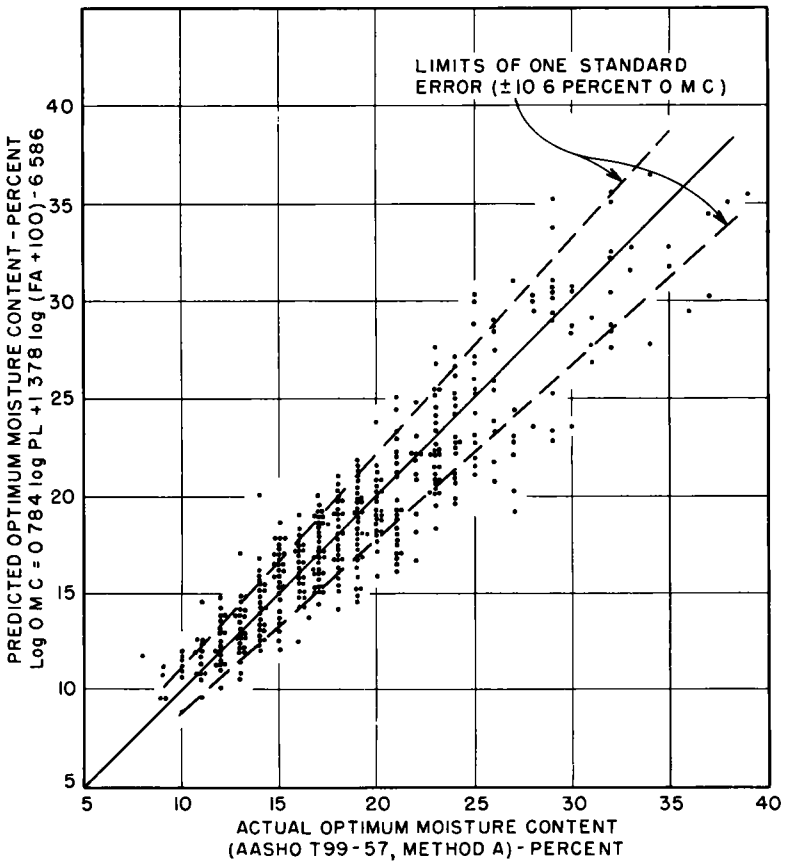

Figure 10. Actual vs predicted optimum moisture content from Eq. 5, analysis 4.

where there might be a restricted range of soil formation processes. The actual optimum moisture contents ranged from 12 to 34 percent. The standard error of estimate was ±1.13 percent moisture.

In regression analyses 6 through 9, the relationship of maximum dry density to several independent variables was investigated. Analysis 6 involved three independent variables that had appeared most closely related to maximum dry density during the examination of simple relationships: liquid limit, plastic limit, and fineness average. The data were used directly in the analysis without adjustment. The standard error of estimate was ±4.44 pcf.

Analysis 7 related maximum dry density to all the independent variables in analysis 6 plus optimum moisture content, which was known from the first study to have a very good correlation with maximum dry density. All of the data were logarithmically transformed and adjusted where necessary. The standard error of estimate for this analysis was ±2.52 pcf.

Analysis 8 related maximum dry density to plastic limit and fineness average, the number of variables being reduced to simplify the predicting equation. Although the number of variables was reduced from that used in analysis 6, the accuracy was slightly improved, probably due to the logarithmic transformations. The standard error of estimate was ±4.32 pcf. The predicting formula developed in analysis 8 is

$$\log MDD = 7.247 - 0.567 \log (PL + 20) - 0.110 \log FA \qquad (6)$$

Figure 11, developed from this formula, shows maximum dry density values for the range of plastic limits and fineness-average values studied.

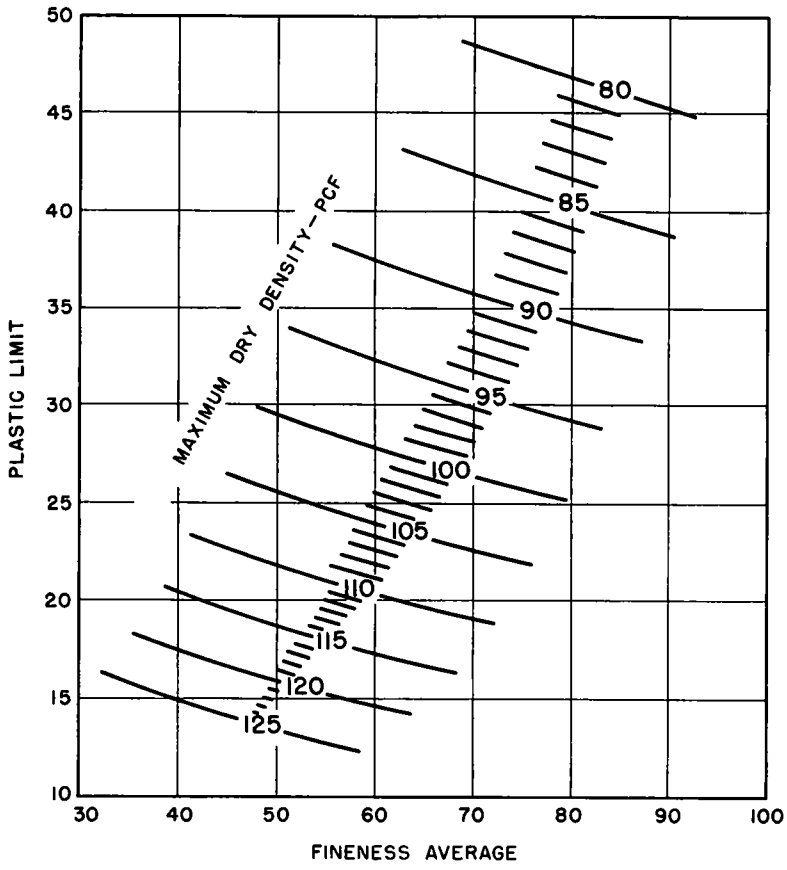

Figure 11. Relation of maximum dry density (AASHO T99-57, Method A) to plastic limit and fineness average, analysis 8.

Analysis 9 was the same as analysis 8, except that the data were limited to the 40 samples from Loudon County, Tenn. for which the maximum dry densities ranged from 83 to 119 pcf. The standard error of estimate was found to be ±2.98, which is smaller than that of analysis 8, and reflects the reduction in soil varieties.

To summarize the results of the regression analyses, the standard errors of estimate given in Table 4 show that the two formulas developed for Loudon County, Tenn., in analyses 5 and 9 fit the data much better than the corresponding formulas developed in analyses 1, 2, 3, 4, 6, and 8 for several states. Analysis 7 included the test value of optimum moisture content as an independent variable and, therefore, should not be considered with the other eight analyses. The standard errors of estimate of the Loudon County formulas were 1.1 percent moisture and 3.0 pcf for optimum moisture content and maximum dry density, respectively. To show the relative magnitude of these standard errors of estimate, they may be compared to the averages of the test values examined. The standard error of 1.1 percent moisture is 5.3 percent $\left(\frac{1.1}{20.94} \times 100 = 5.3\right)$ of the average optimum moisture content (20.94); the standard error of 3.0 pcf density is 2.9 percent $\left(\frac{3.0}{102.3} \times 100 = 2.9\right)$ of the average maximum dry density (102.30).

The standard errors of estimate of the formulas developed in analyses 1, 2, 3, 4, 6, and 8 for all the samples, were approximately 2 percent moisture and 4.4 pcf density. In terms of the average test values, (18.75 percent moisture for optimum and 105.28

pcf for maximum density), these standard errors of estimate are 10.7 percent and 4.2 percent.

It is possible that different combinations of the independent variables from those used in this study could result in better correlations; only those combinations given in Table 4 were analyzed. The selections of variables were based mainly on the data shown in Table 3 and on the partial correlation coefficients developed in the analyses.

COMPARISON OF PREDICTING METHODS

Optimum Moisture Content

To test the formula from analysis 4 (see Fig. 9) for predicting optimum moisture content from plastic limit and fineness average, data for 10 soils varying considerably in characteristics and from widely separated geographical areas were selected from the files. These data are given in Table 6. The predicted and actual moisture contents are given in Table 7. Also given are values predicted by the first Public Roads method (PL and LL) and by methods developed by Jumikis, Turnbull, Davidson and Gardiner, and Rowan and Graham. The results indicate that the PL and FA method is a slightly better predictor than the PL and LL method. The other four methods are at a disadvantage in this type of comparison because they were developed for a more limited range of soil types (origin). Some predictions by each method are quite accurate.

TABLE 6

SOIL DATA USED IN COMPARISON OF METHODS FOR PREDICTING OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY

Location Sampled	Soil[1] Series	Horizon Sampled	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	LL	PL	PI	Specific Gravity	Shrinkage Limit	Shrinkage Ratio	Percent Gradation[2] Finer Than							
											4 7 Mm	2 0 Mm	0 42 Mm	0 074 Mm	0.020 Mm	0 005 Mm	0 001 Mm	FA
Ottawa, Ill	[3]	--	14	119	28	15	13	2 72	13	1 95	100	98	92	78	62	41	21	65
Bayfield Co , Wisc	Golgebic	A_3	14	109	24	22	2	2 71	23	1 68	100	99	90	54	35	13	7	50
Jerome Co , Idaho	Portneuf	B_2	16	107	28	22	4	2.72	19	1 73	100	100	99	97	51	22	13	64
New Castle Co , Dela	Manor	A	17	109	35	26	9	2 71	23	1 61	100	89	77	50	42	30	16	51
DeSoto Co , Miss	Grenada	C_3	18	106	41	22	19	2 70	19	1 73	100	100	100	100	67	30	23	70
Strafford Co , N H	Suffield	C_3	20	107	40	22	18	2 74	21	1 72	100	100	99	98	77	51	32	76
Maricopa Co , Ariz	Mohave	B_3	20	109	43	20	23	2 66	11	1 96	100	99	91	73	50	40	28	64
Elbert Co , Ga	Cecil	B_3	22	100	68	34	34	2 74	26	1 53	100	97	80	68	64	55	42	68
Madison Co , Iowa	Winterset	B_2	25	94	70	30	40	2 73	9	2 02	100	100	99	98	80	52	42	79
Albemarle Co , Va	Davidson	B_2	31	89	72	37	35	2 89	24	1 60	100	100	99	95	89	82	75	90

[1] Sampled and named by Soil Conservation Service, U S Department of Agriculture.
[2] Based on fraction passing No 4 sieve (same fraction used in compaction test)
[3] AASHO Road Test embankment, test data are average values of cooperative materials testing program reported by Shook and Fang (13)

Maximum Dry Density

To test the formula from analysis 8, (see Fig. 11) for predicting maximum dry density from plastic limit and fineness average, data for the 10 soils described in Table 6 were again used. The actual and predicted values are given in Table 8. This summary also gives the densities predicted by the first Public Roads method (PL and LL) and by methods proposed by Davidson and Gardiner and by Rowan and Graham. The values predicted on the basis of plastic limit and fineness average are generally closer to the actual values, although in a few cases one of the other methods has a closer estimate.

Another test of the formulas from analyses 4 and 8 was made using data from Alaska and areas outside the United States. The comparisons of estimated and actual test values are given in Table 9. The soils used in this test are the same, except for the Hawaiian soils, as those used in the first Public Roads study to evaluate the PL-LL method (see Table 2). Estimates for the Hawaiian soils could not be made because the

TABLE 7

COMPARISON OF METHODS FOR PREDICTING OPTIMUM MOISTURE CONTENT

Soil Sample[1]	Optimum Moisture Content (%)						
				Predicted by			
	Test Value[2]	PL and FA	PL and LL	Jumikis	Turnbull	Davidson and Gardiner	Rowan and Graham
AASHO embankment	14	13	14	13	19	12	9
Gogebic	14	15	15	14	13	18	18
Portneuf	16	17	15	15	18	14	14
Manor	17	18	18	21[3]	14	14	15
Grenada	18	18	18	22	20	21	16
Suffield	20	19	18	21	24	23	18
Mohave	20	16	18	23	19	14	7
Cecil	22	25	26	--[4]	22	28	18
Winterset	25	25	24	--[4]	28	18	6
Davidson	31	32	27	--[4]	38	32	21
Sum of deviations	0	13	16	--[5]	28	37	63

[1]Description and basic data in Table 6.
[2]Determined by AASHO Designation T99-57, Method A.
[3]Extrapolated.
[4]Beyond limits of predicting chart (Fig. 1).
[5]Insufficient data.

TABLE 8

COMPARISON OF METHODS FOR PREDICTING MAXIMUM DRY DENSITY

Soil Sample[1]	Maximum Dry Density (pcf)				
		Predicted by			
	Test Value[2]	PL and FA	PL and LL	Davidson and Gardiner	Rowan and Graham
AASHO embankment	119	118	114	118	125
Gogebic	109	110	112	112	109
Portneuf	107	107	112	110	109
Manor	109	104	106	108	111
Grenada	106	106	106	99	105
Suffield	107	105	106	99	108
Mohave	109	110	106	111	125
Cecil	100	92	92	85	105
Winterset	94	94	95	98	126
Davidson	89	87	90	81	100
Sum of deviations	0	20	30	52	76

[1]Description and basic data in Table 6.
[2]Determined by AASHO Designation T99-57, Method A.

TABLE 9

COMPARISONS OF ACTUAL OPTIMUM MOISTURE CONTENTS AND MAXIMUM DRY DENSITIES[1] WITH THOSE ESTIMATED BY FIGURES 9 AND 11, RESPECTIVELY, BASED ON PLASTIC LIMIT AND FINENESS AVERAGE

(SOILS FROM ALASKA AND OUTSIDE THE UNITED STATES)

Soil		Optimum Moisture Content (%)			Maximum Dry Density (pcf)		
Location	Kind	Actual	Est.	Deviation	Actual	Est.	Deviation
Alajuela, Costa Rica	Lateritic	41	32	-9	77	87	+10
Las Lomas, Panama	Lateritic	29	32	+3	88	85	- 3
Liberia	Lateritic	20	21	+1	107	96	-11
		12	12	0	126	120	- 6
		15	14	-1	117	112	- 5
Addis Ababa, Ethiopia	Black Cotton	39	38	-1	80	80	0
Kodiak, Alaska		20	21	+1	103	98	- 5
		24	25	+1	95	93	- 2
		33	30	-3	80	84	+ 4
Kenai-Kasilof, Alaska		10	9	-1	131	130	- 1
		14	15	+1	118	114	- 4
		13	12	-1	124	120	- 4
Sum of deviations				23			55

[1] Determined by AASHO Designation T99-57, Method A.

grain-size analyses were not available. The sums of the deviations shown in Table 9 are 23 percent moisture and 55 pcf density, and are about half of the corresponding values resulting from the methods developed in the first study.

SUPPLEMENTAL INFORMATION

To determine if the deviations of the predicted values from the test values of optimum moisture have a normal distribution, the deviations given in Table 1 were plotted in Figure 12. The resultant curve closely approximates a standard normal distribution curve; this is evidence that the standard error of estimate is a reasonable measure of the accuracy of the predicting methods.

Simple correlation coefficients between pairs of variables used in analyses 1, 3, 6, and 7 are given in Tables 10 and 11. The larger the coefficient, the better the correlation, 1.00 being perfect.

The test data used in the multiple linear regression analyses have been tabulated and are available to other researchers who would like to make additional studies of the data. This 8-page tabulation is available from the Chief, Physical Research Division, U.S. Department of Commerce, Bureau of Public Roads, Washington 25, D.C. In addition to the basic classification and compaction test data for 527 soils, the tabulation includes the location where each soil was sampled, the soil series name, the soil type or textural classification of the A-horizon, and the horizon actually sampled.

SUMMARY

It has been generally known that the optimum moisture content and maximum dry density obtained in compaction tests are related to the plasticity and gradation of the soil material. This report has shown the relationships of the optimum moisture content and maximum dry density to (a) each of several plasticity and gradation characteristics, and (b) two or more plasticity and gradation characteristics, taken jointly.

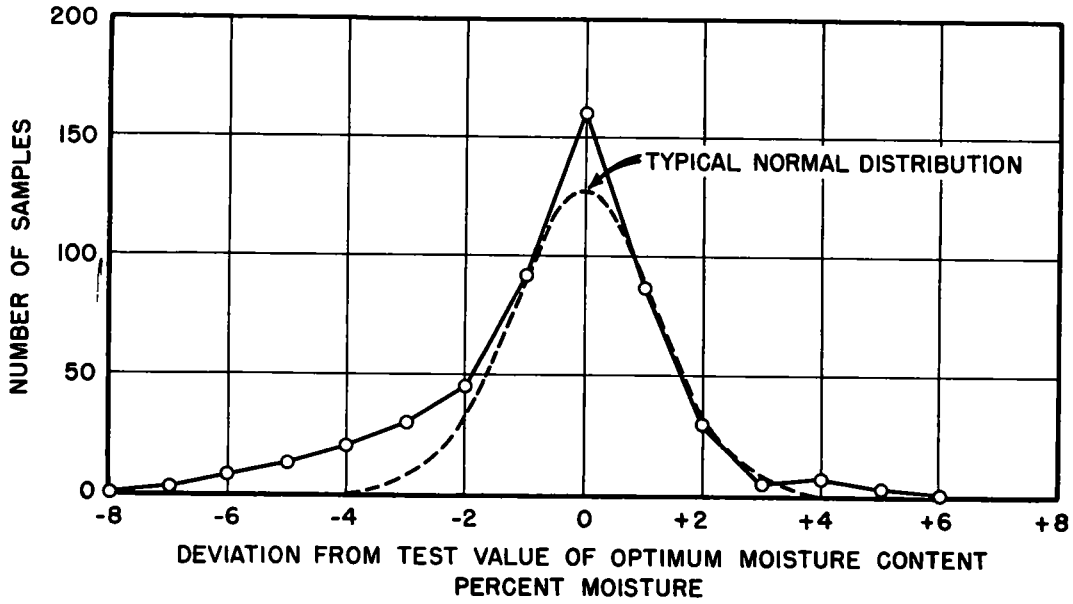

Figure 12. Distribution of deviations of predicted optimum moisture contents, determined by LL and PL method, from actual test values.

TABLE 10

SIMPLE CORRELATION COEFFICIENTS BETWEEN PAIRS OF VARIABLES USED IN OPTIMUM MOISTURE ANALYSES 1 AND 3

Analysis 1

	OMC	LL	PL	PI	D'_{50}	FA	0.001
OMC	1.00						
LL	0.87	1.00					
PL	0.91	0.84	1.00				
PI	0.72	0.95	0.62	1.00			
D'_{50}	0.47	0.45	0.35	0.45	1.00		
FA	0.75	0.77	0.64	0.74	0.73	1.00	
0.001	0.76	0.90	0.66	0.91	0.50	0.84	1.00

Analysis 3

	Log OMC	Log LL	Log PL	Log (PI+15)	Log $\left(\frac{D'_{50}}{100}\right)$	Log (FA+100)	Log (0.001 fract. +40)
Log OMC	1.00						
Log LL	0.89	1.00					
Log PL	0.90	0.83	1.00				
Log (PI+15)	0.74	0.94	0.59	1.00			
Log $\left(\frac{D'_{50}}{100}\right)$	0.76	0.82	0.63	0.81	1.00		
Log (FA+100)	0.77	0.77	0.63	0.74	0.92	1.00	
Log (0.001 fract. +40)	0.76	0.88	0.62	0.92	0.90	0.84	1.00

TABLE 11

SIMPLE CORRELATION COEFFICIENTS BETWEEN PAIRS OF VARIABLES USED IN ANALYSES 6 AND 7 FOR MAXIMUM DRY DENSITY

Analysis 6

	MDD	LL	PL	FA
MDD	1.00			
LL	0.81	1.00		
PL	0.89	0.84	1.00	
FA	0.70	0.77	0.64	1.00

Analysis 7

	Log MDD	Log (OMC+15)	Log LL	Log (PL+20)	Log FA
Log MDD	1.00				
Log (OMC+15)	0.97	1.00			
Log LL	0.81	0.89	1.00		
Log (PL+20)	0.90	0.90	0.83	1.00	
Log FA	0.67	0.74	0.75	0.61	1.00

The multiple correlations based on two or more plasticity or gradation characteristics, provided several methods for predicting the optimum moisture content and maximum dry density as determined by AASHO Designation T99-57, Method A. The simplest of these prediction methods seems to be better than those previously published, when considering a variety of soils from large geographical areas. The best correlations of compaction data with classification data were obtained when the analyses were made on test data from only one county, which was the smallest geographical unit considered.

The predicted values of optimum moisture content and maximum dry density are useful for several purposes. They are useful for (a) determining the amount of water to use in the compaction test for the first moisture-density point; (b) rapidly appraising compaction test results, when the classification data are readily available; (c) reducing the number of compaction tests required in areas where the prediction methods have been proven to be sufficiently accurate for construction control purposes; and (d) denoting unusual soils that are different from those generally encountered and which may cause construction difficulties.

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