HIGHWAY RESEARCH RECORD

Number 177

Symposium on Compaction of Earthwork and Granular Bases

17 Reports

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Foreword

The papers printed in this RECORD will be of interest to both researchers and to practicing engineers concerned with the design and construction of compacted earth structures. The papers present information on various aspects of the compaction problem. They include reports on specification trends and major compaction problems (Johnson, Wahls), available information on the structural properties of compacted soil (Langfelder and Nivargikar), and a laboratory investigation of the rheological properties of compacted soil (Pagen and Jagannath) which leads to a general program to determine the optimum type and amount of compaction energy. A large field study to evaluate typical compactors and rapid control methods (Hampton, Selig, Truesdale) presents data indicating the major effect of moisture on the compaction of soils. These conclusions have a direct application to construction practice. A new laboratory compaction test for granular material (Forssblad) is presented. Papers on rapid nondestructive control tests and methods (Williamson and Witczak, Anday and Hughes, Weber and Smith, Sheeran et al) discuss many aspects in which management of inspection and utilization of modern equipment and methods increase productivity in the compaction of granular base materials and soils. Proposed new testing techniques and evaluation of current compaction and controls methods are also included (Sherman et al, Humphres and Jasper, McDowell, Campen et al).

Contents

Symposium on Compaction of Earthwork and Granular Bases

Introductory Remarks

A. W. JOHNSON, Engineer of Soils and Foundations, Highway Research Board

•ONE PURPOSE of an introduction is to place the subject in perspective with times and places, and with needs that have changed with advances in our state of knowledge. We can do this by viewing the subject from three vantage points: (a) by reviewing our knowledge of compaction, the skills used in its application, and the development of those skills; (b) by assessing our present knowledge and present application in terms of current practices; and (c) by taking a foreglance into the near future to see how we may make even better use of compaction.

In organizing this symposium, your Committee did not look deeply into the origin of modern compaction and the development that followed to learn the scope of engineering involved. That has been done. The results are available in the literature, including several HRB publications (1). However, it may be of interest to note here, as we review the beginnings when O. J. Porter (2, 3, 4) and R. R. Proctor (5) developed moisture-density-compactive effort relationships and related the results of field and laboratory compaction, each developed a system of controls that influenced those properties of soils that needed to be controlled by compaction. Thus they determined not only the optimum moisture content and maximum density for given compactive efforts, but they also related those values to values indicative of swell, bearing value, shear strength, and consolidation of the compacted soils under their own weight in embankments.

After the publication of Proctor's work some of us accepted that part or those parts of Porter's or Proctor's systems that we could fit into our plans and specifications conveniently. Some of us recognized a need for the compaction test. Several highway departments commenced constructing projects under density and moisture control. Some used the Proctor Plasticity "Needle," at least on an experimental basis in the control of construction. Some adopted the California Bearing Value Test to aid in the design and construction of bases and subgrades. In 1938, AASHO standardized the compaction test. In doing this, AASHO accepted the Proctor mold and rammer but changed the compactive effort from 25 firm, 12-inch strokes to 25 blows of the rammer dropping a distance of 12 inches.

Those in positions of authority felt that it was not good engineering to specify that contractors compact soils to 100 percent of a density value that had become known erroneously as "the maximum density." This seemed an unusual decision, for there were few, if any, instances in the specifications that required a reduced percentage of some predetermined value believed to be desirable. Placing both an upper and a lower limit on percent density as in specifications for certain other requirements in road building was believed unworkable for compaction, and there were few of us at that time who were willing to admit that we were capable of designating the proper lower limit of percent density for all types of soil and their uses in different parts of the road structure.

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Thus, while the compaction test was soon standardized, few accepted the other accompanying test methods employed by either Porter or Proctor, nor did we replace them soon with other acceptable tests to aid in the proper use of compaction.

The foregoing concerned the beginnings of scientific methods for use and control of compaction. Since that period a number of organizations have developed systems that relate moisture and density control to some soil property or properties that they desire to control in design and construction. One example of a system of this nature is described in a paper that is a part of this symposium.

Assessments of our present knowledge are being made from time to time. In one sense these become our hindsights, but only when they are brought into focus with previous studies and both are interpreted in the light of what we know today.

In recent years several summaries have been made of current practices concerning the use of compaction as indicated by limits in state highway standard specifications. Some reports on methods of designing flexible type pavements have indicated the use of compaction as a design tool. However, those summaries of current practices could well have probed more deeply into both the how and why of the use of compaction. This symposium includes one paper on current practices. It is based on a comprehensive study of the state of the art that has been completed recently. It is important that each of us learns the extent of the differences that exist in the present application of compaction and why those differences exist. When we have answered those questions we may see better what we may gain from the third view in our perspective, namely that of looking into the future possibilities of compaction.

Our state of knowledge regarding the manner and extent in which the type and degree of compaction influences soil properties has increased greatly in recent years. Much of this newly gained knowledge has come from results of laboratory studies. I feel rather strongly that we have inadequate confirmation from field experience that can tell us how to translate this knowledge into behavior of those elements that make up the total road structure. I feel this is true for the variables of soil type as well as for the attendant climatic, load, time and other conditions as they affect or are affected by the degree of moisture-density control and the location and dimensional aspects of the zones involved. More field data are needed on soil responses to the range of conditions that exist. When those data become available, we should be able to use compaction to result in even greater benefits than is now possible.

I hope that those of you who use compaction will take advantage of this symposium to determine (a) if you are using compaction to the greatest advantage; (b) if you can predict with reasonable accuracy the limiting values of shear strength, consolidation, swell, or other soil property that is needed in designing and building each element of the total road structure and how each is dependent upon compaction; and (c) if you have the means for determining the limiting values of moisture content, density, pulse velocity, or the means for applying statistical or any other methods that may be of value in controlling construction satisfactorily. Some progress has been made toward these goals, but it appears that as engineers and scientists we have little more than begun to place ourselves in a position where we can use compaction to its best advantage in providing stable slopes; minimizing post-construction settlements; controlling swell and shrink of subgrades, shoulders and slopes, erosion of slopes, detrimental effects of freezing and thawing, detrimental compaction by traffic under in-service conditions; and in controlling uniformity to provide pavements that do not result in a pitching, rolling or "choppy" ride that should not be characteristic of our road surfaces today.

The idea of using compaction as an engineering tool should no longer need to be "sold." However, because compaction does so markedly influence the behavior of the soil, there is indeed a strong need to further the understanding of its influence on soil behavior. There is also a need to continue to work for better means for its control during construction. All of these items are keys to providing a stronger, better riding, and safer riding surfaces.

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Some Factors Influencing Shear Strength and Compressibility of Compacted Soils

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•THE GREAT majority of state highway departments are presently using dry density as the principal criterion for judging the quality of compacted earthwork. This criterion implies that increased dry density produces improved engineering properties in the material. Although the use of dry density for field control can be easily accomplished, particularly with the increasing use of nuclear devices, its value as a usable criterion is only valid insofar as the dry density does, in fact, indicate the engineering properties of the material. The two most important and generally applicable properties that concern the highway designer are the shear strength and the compressibility characteristics of the compacted materials. However, for certain soils and in many geographic areas the shrinkage and swell potential and the frost susceptibility may be of greater concern to the highway designer than the shear strength and compressibility characteristics.

The major purpose of this paper is to present a review of the presently available (1966) literature concerning some of the factors, in addition to the dry density, that affect the engineering behavior of compacted soils. Because of space limitations only shear strength and compressibility will be dealt with; for clarity of presentation, cohesionless and cohesive materials will be discussed separately.

SHEAR STRENGTH OF COHESIONLESS MATERIAL

Major Factors

The shear strength of cohesionless materials is essentially controlled by five factors: (a) mineralogical composition, (b) size and gradation of the individual particles, (c) shape of the individual particles, (d) void ratio or dry density, and (e) confining pressure. Assuming that the shearing resistance can be expressed by the Coulomb failure criterion with zero cohesion, the first four factors mentioned affect the angle of internal friction, whereas the fifth factor controls the normal stress. The first three factors are properties of the material and therefore the choice of material should be based on a consideration of these properties. The confining pressure is prinicipally governed by the amount of overburden that exists above the compacted material. Increased confining pressures for a given cohesionless material will not only produce larger shearing resistance but will affect the stress-strain behavior of the material. The magnitude of the confining pressure also affects the dilation characteristics and consequently affects the shearing resistance. It is, therefore, only the dry density or void ratio that can be significantly changed during the compaction process. The first four factors will be discussed briefly in the following sections.

Size and Gradation Effects

Holtz and Gibbs (1) performed a series of triaxial tests in order to study the effect of the maximum particle size on the shearing resistance of a sand-gravel mixture.

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For a 20 percent gravel and 80 percent sand mixture there appeared to be a slight increase in shear strength when the maximum size was increased from 3/4 in. to 3 in. For the same materials with a 50 percent gravel and 50 percent sand mixture there was essentially no difference in shearing resistance as the maximum size was increased.

Kolbuszewski and Frederick (2) performed shear tests using different sizes of glass beads. For the rather limited range of median size of 0.48 mm to 0.86 mm, it was found that with increasing grain size the shearing resistance first decreased and then increased in strength over most of the relative porosity range.

Kirkpatrick (3) performed triaxial shear tests on a sand of very uniform particle size, ranging from 0.3 mm to 2 mm. Microscopic examinations were performed to insure uniformity of shape and mineralogy, and the sand was fractionated into six sizes. The results indicate that for equal relative porosities the angle of internal friction decreases as the mean particle size increases, when no energy correction is applied to the data. If, however, the angle of internal friction is not determined from the peak point on each stress-strain curve but rather at the strain where the sample attains a minimum volume, then the frictional component of the angle of internal friction thus obtained appears to be essentially independent of grain size. These data imply that the effect of grain size is to modify the stress-dilation characteristics rather than the actual frictional resistance of the material. This is consistent with Skempton's (4) hypothesis that the contact stresses that exist in a stressed mass of soil approach the yield strength of the grains. Therefore, the contact stresses should be independent of the grain size and the frictional component of the shearing resistance will be independent of the size of the grains. However, for practical considerations the work required to cause volume change must be considered together with the work required to overcome frictional resistance.

Considering all of these data, it appears that the effect of grain size on the frictional resistance has not been definitely established, although it appears that the effect on the dilation characteristics causes variation in the shearing resistance.

The principal influence of gradation characteristics is the effect it has on the limiting porosity of a given material. A more well-graded material will have a lower minimum porosity and, because shear strength is inversely related to porosity, a more well-graded material will have a larger shear strength for any relative porosity than a more poorly-graded material. The direct effect of gradation can be obtained by replotting Kirkpatrick's data for these sand mixtures in terms of absolute porosity vs angle of internal friction. Although the variation in gradation is not very large it appears that the better-graded material exhibits a slightly lower angle of internal friction than the more poorly-graded material. However, for similar compaction procedures a well-graded material will obtain a smaller porosity than a poorly-graded material and will, therefore, exhibit a larger shearing resistance.

Shape and Surface Texture Effects

The shape of individual particles has long been recognized as a factor influencing the shearing resistance of a granular material. Terzaghi and Peck (5) have indicated that for a granular material with round, uniform grains the angle of internal friction varies from about 28.5 deg with material in a loose state to about 35 deg for the same material in a dense state; the corresponding values for angular, well-graded soils are 34 and 46 deg respectively. The range of these values is so well accepted in practice that the effect of angularity is generally used to estimate approximately the limiting values of the angle of internal friction for a given relative density.

Data by Holtz and Gibbs (1) on an angular quarry material and a river deposit material at the same relative density (70 percent) exhibit angles of internal friction of 40 and 38 deg respectively. Holtz (6) states that a reasonable range of angle of internal friction might be 22 to 45 deg for rounded sandy soils at low and high relative densities and 27 to 52 deg for angular gravelly materials at low and high relative densities. Morris (7) presents some interesting data on a 1/8-in. maximum size crusher-run basalt. An attempt was made to separate shape effect from surface texture effects. These data indicate that merely rounding the particle shape without altering the texture results in an increase in shear strength, and smoothing the surface texture without altering the particle shape results in a reduction in shear strength. Unfortunately, density data are not presented and therefore it is quite possible that modification to the material resulted in density changes that affected the data. In general, however, naturally occuring rounded material will also exhibit a smooth surface texture and angular material will have a rough texture so that it is not usually necessary to consider these factors separately.

Void Ratio or Dry Density Effects

As previously stated, for a given material it is only the void ratio or dry density that can be modified by the compaction process. For a given cohesionless material it appears that the shear strength is directly related to the density but is independent of the compaction process used to obtain this density. Data presented by Means and Parcher (8) indicate that for a particular granular material the angle of internal friction is inversely related to the void ratio. The change in the angle of internal friction with a change in void ratio appears to differ somewhat depending on the soil being tested—varying from 2 deg for silty sands to about 6 deg for uniform gravels for a 0.1 change in void ratio. Zolkov and Wiseman (9) have presented similar data on dune and beach sands that indicate an increase of about 4 deg for a decrease in void ratio of 0.1.

Wu (10) investigated the effect of initial void ratio on the angle of internal friction by using uniform sands with mean diameters of 0.15, 0.44 and 1.00 mm. The angle of internal friction for a given void ratio was different for each material; however, the change in the angle of internal friction increased by about 2 deg for a 0.1 decrease in void ratio. When these same data are plotted in terms of the angle of internal friction vs the compacted relative density, the relationship collapses to a unique association independent of grain size. The angle of internal friction increases about 1 deg for a change of 0.1 in relative density. These data are consistent with the previous data with the exception that the mean grain size appears to affect the angle of internal friction at a given void ratio.

COMPRESSIBILITY OF COHESIONLESS MATERIAL

The compressibility characteristics of compacted cohesionless materials are primarily influenced by the same factors that influence the shear strength, namely, the mineralogical composition, size and gradation of the particles, shape of the particles, void ratio and confining pressure. In general, the compressibility decreases with increasing gradation, decreasing as-compacted void ratio, decreasing angularity, and increasing confining pressure.

The mineralogy of the individual particles contributes to the compressibility characteristics by influencing other properties such as the size, shape, cleavage planes, elasticity, etc., of the particles. Compression tests on sand-mica mixtures performed by Gilboy (11) showed that compressibility increases as the percentage of plate-shaped particles increases. McCarthy and Leonard's (12) investigation on micaceous sands and silts also indicated that the compressibility is significantly affected by the percentage of mica that is present in the material. The presence of plate-like particles, such as mica, produces two effects that influence the compressibility. First, the surface properties of these layer-latticed minerals are probably smoother than the massive-shaped minerals and therefore can be more easily densified. Second, the introduction of these flat particles produces a decrease in the compacted density which also contributes to an increase in compressibility.

Wu (10) has presented data to indicate that decreasing grain-sized material will exhibit increasing compressibility. These data are for samples with mean diameter from 0.51 to 1.00 mm. Using the same method of compaction, the initial void ratio increases with decreasing grain size; however, the initial relative density increases with decreasing grain size. Therefore, at a constant relative density, the increase in compressibility would be even more pronounced than indicated. Burmister (13), working with materials ranging from gravelly sand to silty fine sand, also found that at constant relative density (40 percent) the compressibility increased with increasing fineness of the material.

The influence of relative density on compressibility is similar to the effect of relative density on shear strength; that is, increasing relative densities for a given material will cause decreasing compressibilities. Gardner (14) presented such data on Atlantic City beach sand for a range of initial relative densities from 27 percent to approximately 100 percent and over a stress range from $1/8$ ton/sq ft to 55 ton/sq ft.

Schultze and Menzenbach (15) have presented data on 25 clean dry sands indicating that compressibility increases with increasing initial void ratio. These data also indicate that the compressibility increases for a given initial void ratio as the void ratio between the maximum and minimum states increases. These increasing compressibilities are due to the properties of the material such as grain shape and grain-size distribution.

The grain shape appears to have two effects. First, more angular grain shape decreases the compacted density that can be obtained and second, it decreases the stress required to cause crushing of grains. The crushing of grains causes degradation of the material and nonelastic densification of the materials.

SHEAR STRENGTH OF COHESIVE SOILS

The shearing strength of a compacted cohesive soil is primarily affected by the water content, gradation, dry density, soil structure, thixotropy and the normal effective stress acting on the failure plane. The water content that influences the shear strength is not only controlled by the molding water content, but includes any changes in moisture conditions that occur after placement. The dry density is controlled by the amount of compactive effort expended during compaction, the water content at which compaction takes place, the method used to compact the soil and any density changes that occur after initial compaction. The soil structure[1] is controlled by the method of compaction used and the water content relative to the optimum water content. The thixotropic effects for a given soil depend upon the time allowed for strength changes to occur and the strain level at which strength is defined. The effective stress that acts on an element of soil is produced by external pressure, such as overburden, and internal pressure exerted by the apparent negative pore water pressure. The overburden pressures on subgrades are quite small; therefore, the major contribution to the effective stress would be the internal pressure.

Influence of Effective Stress

The shear strength of compacted cohesive soils can be interpreted in terms of effective or total stresses in the same manner as saturated soils; however, the determination of the effective stress in a compacted soil is complicated because of the three-phase nature of the system. Because of this complication the shear strength of compacted soils is generally investigated in terms of total stress unless the test specimen is soaked prior to testing and pore pressures are measured during shear. Nevertheless, it is the application of an effective stress and not a total stress that causes an increase in shearing resistance of a compacted cohesive soil.

The shear strength of a compacted cohesive soil cannot, in general, be determined from the well-known Terzaghi equation because the pressure in the gas and water phases of the soil may be considerably different. Bishop (17) proposed the following expression for defining the effective stress in an unsaturated soil:

$$\bar{\sigma} = \sigma - Xu_w' - u_a (1 - X) \qquad (1)$$

[1]Lambe (16) defines soil structures as "the arrangement of particles and the electrical forces acting between them."

where

$\bar{\sigma}$ = effective stress;
σ = total stress;
u_a = pore air pressure;
u_w = pore water pressure; and
X = a factor depending primarily on the degree of saturation, but which may also be influenced by stress history, wetting or drying sequence, and soil types.

The solution to this expression requires a knowledge of X, u_a and u_w. The pore air and pore water pressures can be determined using modifications of the pressure plate procedure (18). The determination of the X-factor requires the testing of duplicate samples of saturated and unsaturated specimens and the assumption that the angle of internal friction remains constant upon saturation.

Assuming the Bishop equation adequately describes the effective stress, it is possible to obtain a qualitative estimate of the change in effective stress along a compaction curve. On the dry side of optimum water content the air permeability is high and therefore the pore air pressures produced by compaction should be rapidly dissipated. At optimum and slightly wet of optimum, although the air permeability is quite small, the X-factor is large and therefore the term u_a (1 - X) should be small compared to u_w X in Eq. 1. Assuming the u_a (1 - X) term can be neglected, Eq. 1 degenerates to

$$\bar{\sigma} = \sigma - Xu_w \qquad (2)$$

For a constant value of total stress, the effective stress becomes a function of the X-factor and u_w. Assuming the X-factor is only related to degree of saturation and the pore water pressure is related to the water content similar to the data presented by Lambe (19), Bishop and Blight (20) and Olson and Langfelder (21), Figure 1 schematically represents the relationship of dry density, X-factor and pore water pressure to the molding water content.

Figure 1 shows that on the dry side of optimum, u_w becomes less negative as molding water content and dry density increase but X continuously increases; therefore the effective stress may either decrease or increase depending on the interaction of these two factors. This implies that increased dry density does not necessarily result in increased effective stress.

On the wet side of optimum, the degree of saturation is essentially constant beyond optimum water content and thus X is essentially constant. However, u_w continues to be increasingly less negative as molding water content increases. This implies that the effective stress must decrease on the wet side of optimum.

To estimate the change in shear strength along a compaction curve requires a knowledge of the change in frictional

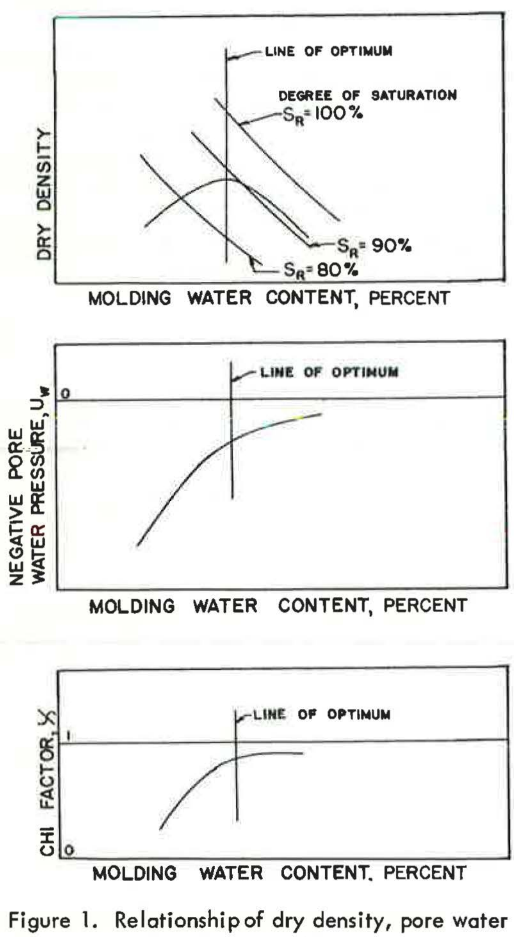

Figure 1. Relationship of dry density, pore water pressure, and X-factor to molding water content.

resistance as well as the change in the normal effective stress on the failure plane. This change in frictional resistance will vary along the compaction curve and therefore it is not possible to establish the change in shearing resistance along the compaction curve from a consideration of effective stress alone.

Effect of Molding Water Content and Soil Structure

Varying the molding water content of a compacted cohesive soil will have an effect upon (a) the initial soil structure, (b) the magnitude of the initial pore water pressure, (c) the dry density of the material, (d) the swelling characteristics, and (e) pore water pressures developed during shear. Each of these factors will, in turn, influence the shear strength of the material.

The initial soil structure of a compacted cohesive soil is governed by the molding water content and the method of compaction. It has been shown (22) that on the dry side of optimum the soil structure will generally be flocculated regardless of the compaction method, but on the wet side of optimum water content the compaction methods producing large shearing strains will produce dispersed soil structures. Increasing degrees of dispersion at water contents wet of optimum are produced by the static method, dynamic method and kneading method respectively. This relationship has been so widely accepted that it is common to associate a dispersed soil structure on the wet side of optimum water content with kneading compaction, and a relatively flocculated soil structure on the wet side of optimum water content with a static type compaction. It appears that a cohesive soil with flocculated soil structure will exhibit a higher as-compacted shear strength than a soil with dispersed structure because of the more rigid nature of the soil skeleton and the reduced pore water pressure developed at low strains.

The influence of induced soil structure on the resulting shearing resistance is also evidence by the behavior of as-compacted soils at different strain levels. At small strain levels the initial soil structure still influences the shearing resistance and therefore the flocculated structure that occurs on the dry side of the optimum water content produces larger shear strength than if the material had a dispersed soil structure. At large strain levels the initial soil structure is essentially destroyed and does not affect the shear strength.

Because the soil structure is an extremely difficult property to measure for clay-sized particles, it is usually the practice to infer the soil structure from other measurable properties. For example, Mitchell, Hooper and Campanella (23) have shown that for essentially the same water content vs dry density curve there is a distinct difference in water permeability on the wet side of optimum water content for different compaction methods. Seed, Mitchell and Chan (22) have presented similar data in terms of shear strength and stress-strain relationships for dynamic, kneading and static-type compaction. The soil structure at low water contents is flocculated because of insufficiency of the water available for formation of the double layer and the absence of interference of the adsorbed water films, and the attraction of the negatively charged surfaces of the clay for the positively charged clay edges and any other cations present. As the water content increases, there is a tendency for greater interference of the water films and if an opportunity for particle rearrangement exists, the soil will tend toward a more dispersed structure. Kneading and dynamic methods of compaction provide this opportunity for particle rearrangement. Therefore, as the molding water content is increased it should be expected that the shear strength should decrease based upon only a change in structure.

As previously noted, an increase in molding water content will produce a less-negative value of pore water pressure which, at least on the wet side of optimum water content, will cause a lower effective stress because of the increased degree of saturation and, hence, the X-factor is essentially constant for increasing water contents on the wet side of optimum. This decrease in effective stress should produce a decrease in shear strength providing the other factors that influence the shear strength are held constant.

The dry density of a compacted cohesive soil is, of course, greatly affected by the molding water content at which the soil is compacted. The dry density is a factor

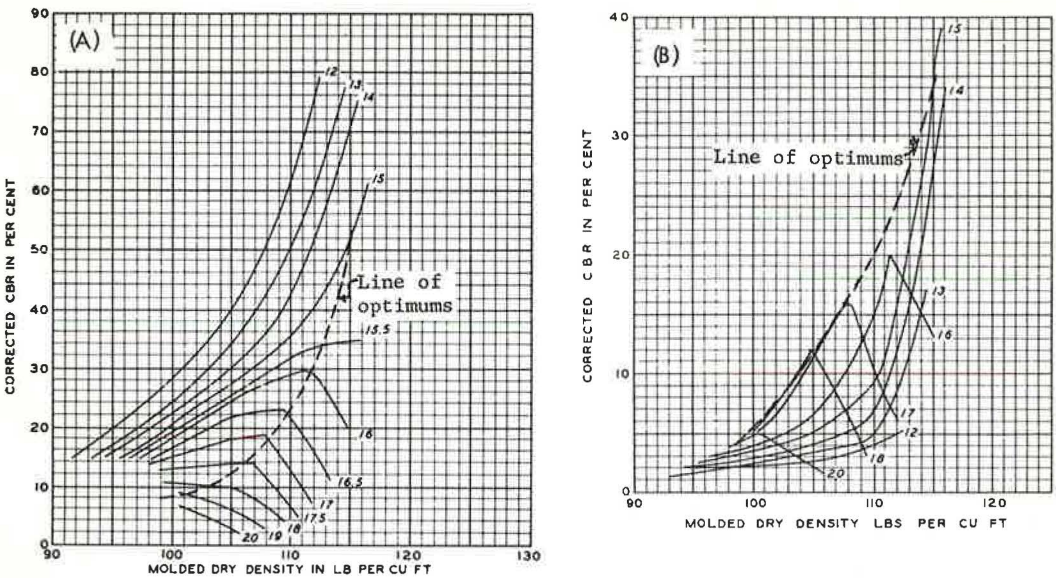

Figure 2. Relationship of molding water content, dry density and strength; (A) unsoaked, (B) soaked (after Turnbull and Foster, 1958).

influencing the shear strength of a cohesive soil, although, as will be shown subsequently, the available data appear to indicate that increasing the dry density will not always produce an increased shearing resistance. Various compaction theories (24, 25, 26, 27) have attempted to define the mechanism by which the molding water tends to affect the dry density that can be obtained by a specific compaction technique. Although these investigators have approached the question from different viewpoints, it is generally agreed that the addition of water to a dry cohesive soil first allows the particles to be more easily packed (up to optimum). After optimum water content is reached, the addition of more water acts to displace soil particles. For soils that exhibit distinct double peaks, Olson (27) suggests that the mechanism producing the first and lower peak may be different from the mechanism producing the upper and most generally recognized peak.

Considering the as-compacted state of a cohesive soil, all the available data indicate that for any constant value of dry density the shear strength will decrease with an increase in molding water content. In fact, CBR data from a series of Waterways Experiment Station publications (28, 29, 30) indicate that for water contents up to approximately 10 percent dry of optimum the strength in almost all cases decreases or remains essentially constant with increasing molding water content, even though the density increases with increasing water content on the dry side of optimum. These data imply that if increased strength is the primary engineering property sought it would be advantageous to compact the soil well dry of the field optimum water content. This would be particularly the case where the natural water content is less than the optimum water content and water must be added.

The available data on the shear strength of compacted cohesive soils that are soaked prior to testing indicate that soils compacted well dry of optimum do not retain high shear strength upon soaking. The soaking of a compacted cohesive soil not only increases the degree of saturation and water content, but may also decrease the dry density of the soil unless a sufficient confining pressure is applied to counteract the possibility of swelling. Seed, Mitchell and Chan (22) have presented data to indicate that for a sandy clay the amount of swelling that takes place because of soaking decreases as the as-compacted water content increases. These data indicate that upon soaking, the final water content is at a minimum and, therefore, the final dry density is at a maximum for a sample that had an initial water content slightly wet of optimum. Data (Fig. 2) presented by Turnbull and Foster (31) for a lean clay indicate that there is a considerable reduction in CBR values, particularly on the dry side of optimum water content after soaking with a surcharge equivalent to the expected overburden pressure. This type of soaking will, in general, allow swelling to occur during the soaking period. The maximum soaked CBR value for any given dry density occurs at approximately optimum water content.

Seed and Chan (32) have presented data on a silty clay (Fig. 3) and an expansive sandy clay both soaked under a low (1 psi) surcharge and tested unconsolidated-undrained in a triaxial apparatus at 1 kg/cm^2 confining pressure. For the silty clay, it appears that the soaked strength is essentially independent of the initial water content for strength defined at large strains, but is dependent on initial water content for strength at low strains. The soaked strength for the expansive sandy clay, for any given density, increases with increasing molding water content at both low and high strains. This difference in the effect of the initial water content on the strength after soaking can be attributed to the swelling potential of the different soils and the strain level at which the strength is defined. Initial flocculated structure that occurs at smaller water contents produces larger swelling potentials than those associated with dispersed structure occurring at high water contents. The effect of the molding water content on the soaked strength of a cohesive soil depends on whether the increased strength at low strains caused by a flocculated soil structure is sufficient to counteract the decrease in strength caused by a greater amount of swell and, therefore, the larger void ratio. For the silty clay, the swelling potential is small and, therefore, there is not a greater tendency for swelling on the dry side of optimum water content than on the wet side of optimum water content. Thus, the influence of the initial soil structure causes the strength at low strains to be larger on the dry side of optimum water content for a given dry density. At large strains, the soaked strength of the silty clay is essentially independent of initial water content because the initial soil structure is destroyed. For the expansive sandy clay, the soaked shear strengths at both small and large strains are dependent on the initial water contents. This is caused by the fact

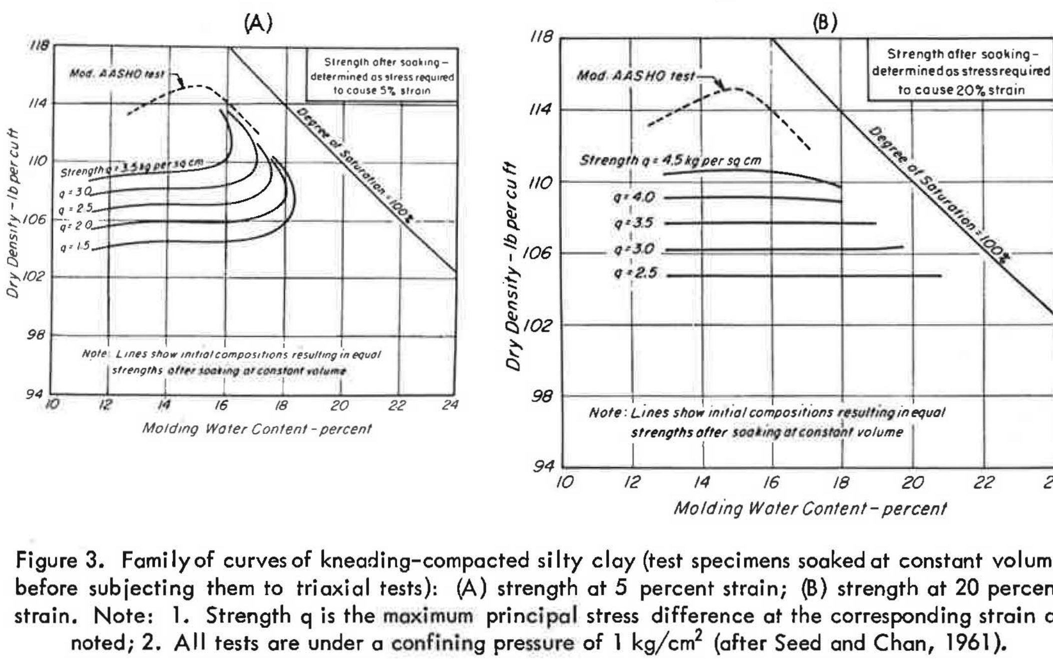

Figure 3. Family of curves of kneading-compacted silty clay (test specimens soaked at constant volume before subjecting them to triaxial tests): (A) strength at 5 percent strain; (B) strength at 20 percent strain. Note: 1. Strength q is the maximum principal stress difference at the corresponding strain as noted; 2. All tests are under a confining pressure of 1 kg/cm^2 (after Seed and Chan, 1961).

that the greater tendency of the flocculated structure to swell counteracts the increased strength associated with a flocculated structure.

If the surcharge during soaking is sufficient to prevent swelling then it appears that the maximum strength at a given dry density occurs at approximately optimum water content. This is consistent with the fact that the shear strength of a compacted cohesive soil at large strains is inversely related to the void ratio.

The development of pore water pressure during the application of a shearing stress will also depend on the molding water content because of its influence on the soil structure. For cohesive soils compacted dry of optimum, the flocculated structure will develop smaller positive pore water pressures at low strains than the small soil compacted wet of optimum. At large strains the initial flocculated structure is destroyed and the pore pressures tend toward the same value. Other factors being equal, this equalization of pore water pressure will cause the soil to exhibit approximately the same shear strength at large strains.

Effect of Dry Density

The changes in shear strength that are produced as a function of changes in dry density alone can be determined by using several different compaction energies and comparing the strengths at a constant value of molding water content. This procedure assumes that there is no effect of possible changes in soil structure as the optimum water content decreases with increasing compaction energy. As previously noted, this assumption should be valid if the strength is measured at large strains; however, if the strength is measured at low strains, the influence of changes in soil structure should not be neglected. In considering the influence of dry density on shear strength it is also necessary to make a distinction between soaked and unsoaked strengths. Finally, it

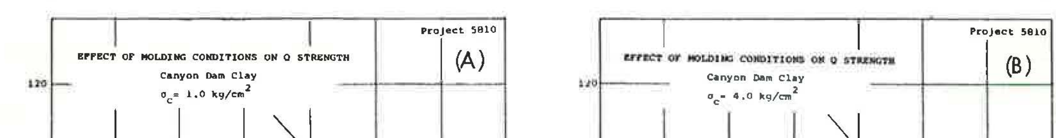

Figure 4. Family of curves for kneading-compacted Canyon Dam Clay (unconsolidated-undrained tests on as-compacted specimens); (A) confining pressure = 1 kg/cm^2; (B) confining pressure = 4 kg/cm^2 (after Casagrande and Hirschfeld, 1962).

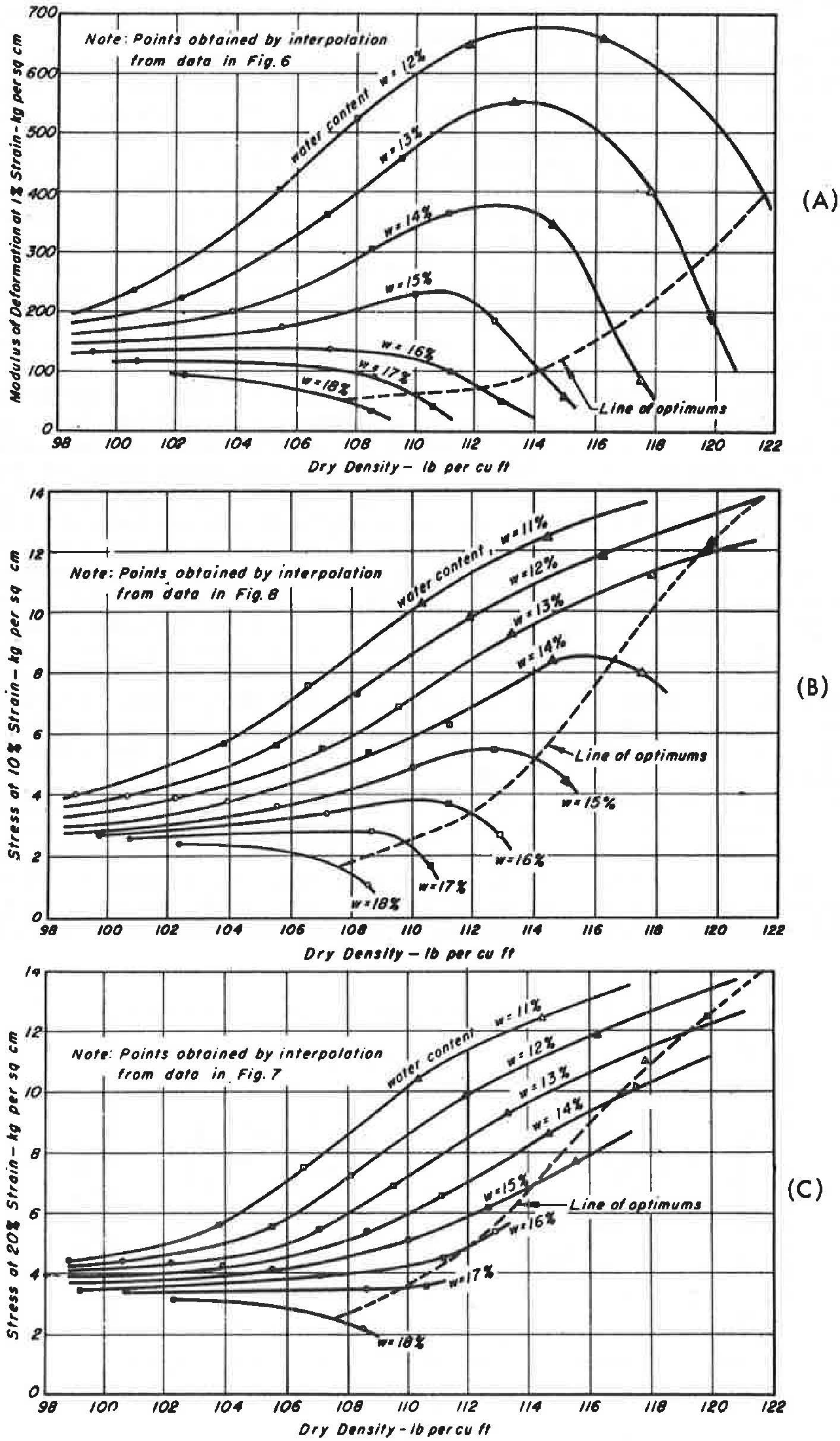

Figure 5. Strength as a function of strain (after Seed and Monismith, 1954).

appears that the method of compaction influences the response of the shear strength to change in dry density at constant molding water content.

Seed and Monismith (34), Seed, Mitchell and Chan (22) and Casagrande and Hirschfeld (35) all have presented data on the relationship between dry density and shear strength at different molding water contents. All these data indicate that an increase in dry density will cause an increase in shear strength for a given water content, provided the shear strength is defined at both large strains (Fig. 3B) and moderate confining pressure (Fig. 4A). In general, the rate of increase in shear strength with an increase in dry density is largest for the lowest value of water content. As the molding water content increases the increase in shear strength is smaller to nonexistent, depending on the soil being investigated. If the stress mobilzed at low strains is plotted against dry density for constant values of water content on soils compacted by different methods of compaction, it can be shown that the relationship between stress and dry density depends on the water content and the method of compaction. For a moderate confining pressure (1 kg/cm^2), statically compacted samples exhibit an increase in shearing resistence with density regardless of the strain level at which the strength is defined. However, for kneading-compacted samples there is a marked change in the relationship between dry density and developed stress as the water content increases. Figure 5 shows data by Seed and Monismith (34) for unconsolidated undrained triaxial tests at 1 kg/cm^2 confining pressure for kneading-compacted Vicksburg silty clay. These data are somewhat typical for kneading-compacted soils and indicate the effect of water content and strain level on the relationship between dry density and developed stress. It can be seen that the decrease in stress for the higher densities with an increase in dry density is most pronounced for the 1 percent strain data and, except for the very wettest water contents, nonexistent for the 20 percent strain data. This is consistent with the conclusions presented earlier, that kneading compaction will produce a flocculated structure on the dry side of optimum water content and a more dispersed structure on the wet side of optimum, and that the flocculated structure is more rigid than the dispersed strucutre. At the lower strain levels the initial structure still influences the strength, whereas at the larger strains the initial flocculated soil structure is essentially destroyed.

It is interesting to note that both field (sheepsfoot or rubber-tire rollers) and laboratory compacted CBR data exhibit relationships between strength and dry density similar to the relationships found at low to medium strain levels for kneading-compacted specimens tested in the triaxial apparatus. Figure 6 shows data reported by Turnbull and Foster (31) for a lean clay compacted by rubber-tired roller and tested using a CBR piston. It can be seen that at approximately the line of optimum for this soil there

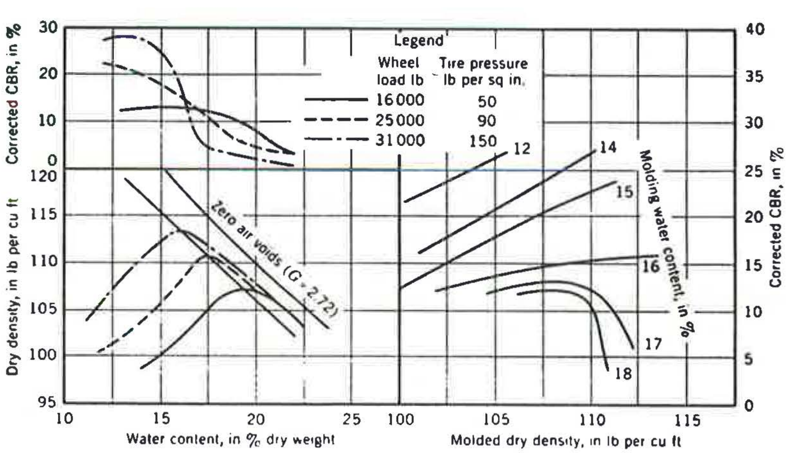

Figure 6. Relationship of CBR, density, and water content for lean clay test fills, 4 coverages, rubber-tired rollers (after Turnbull and Foster, 1958).

is a change from an increase in strength to a decrease in strength for increasing dry densities.

Casagrande and Hirschfeld's (35) unconsolidated undrained strength data on as-compacted clay tested under a large confining pressure indicate that the strength is dependent on initial water content, but is essentially independent of the dry density except at very low degrees of saturation. The difference between these data and the data at moderate confining pressure is that for these tests the confining pressure that was applied was sufficiently large to cause essentially complete saturation of the soil except at the very low degrees of saturation. Therefore, the material essentially behaved as a saturated clay and the shear strength merely depended upon the water content.

The relationship between the shear strength after soaking and the initial dry density depends on the amount of swelling that takes place during the soaking, the compaction method used, and the soil type. Seed and Chan (32) have shown, however, that the soaked strength of a compacted cohesive soil will increase with an increase in initial dry density, regardless of the compaction method, soil type (although the soils investigated were limited), amount of swelling during soaking and strain level. An exception to this conclusion is if strength is defined at low strain and the soil is compacted by a method that produces large shearing strains. For this condition it is possible to obtain a decrease in strength with increasing dry density. The standard laboratory CBR test is performed on a dynamically compacted specimen and the CBR value is obtained at what appears to correspond to a low strain level. Decreases in CBR values for increasing densities at constant water content have been reported extensively in the literature. This same condition also may exist in the field where a subgrade has been compacted by sheepsfoot roller and then soaked during spring thaw, and only small deformations are tolerable before loss of support to the pavement causes a failure.

Thixotropic Considerations

The process of strength changes with time at a constant water content is generally referred to as thixotropy in soil mechanics literature. This property is important when attempting to predict field strengths at some time after compaction from laboratory tests that are generally performed soon after compaction or soaking has been completed.

Mitchell (23) has hypothesized the cause of thixotropy as being the creation of a new equilibrium condition resulting from the cessation of external compaction forces. In order to obtain increases in shear strength with time it is necessary that the final equilibrium condition be conducive to a flocculent structure and the structure immediately after compaction be a relatively dispersed structure. This condition can be produced in certain soils by using kneading compaction methods even up to water contents slightly wet of optimum. In conjunction with this change in soil structure it was found that the initial pore water decreases during aging and also the pore water pressures developed during shearing are smaller for aged samples. It is quite likely, therefore, that there is an increase in strength in terms of total stress but the strength remains constant in terms of effective stress.

In addition to the influence of the molding water content on the amount of strength gain, the strain at which failure is defined also determines the measured amount of strength increase. This is consistent with the previous discussions that indicated that the flocculated soil structure is destroyed at large strains. Therefore, the change in structure, with time, from dispersed to flocculated which produces the larger strengths (either because of a more rigid structure or decreasing pore water pressures, or both) is not effective in producing increased strengths at large strains.

Methods for predicting thixotropic strength gains from index-type tests are not available at the present time. Furthermore, it does not appear to be satisfactory to extrapolate thixotropic behavior of field-compacted soils using laboratory compaction procedure because of possible differences in the soil structure produced by these different compaction methods. However, an awareness of the phenomenon will lead to a better understanding of the behavior of the field-compacted materials that possess this characteristic.

COMPRESSIBILITY CHARACTERISTICS OF COMPACTED COHESIVE MATERIALS

The compressibility characteristics of cohesive materials are significantly influenced by soil type, molding water content, dry density, degree of saturation, and the compaction method. The amount of compressibility for a given range of pressure is influenced by the combined effect of these factors. In general, the compressibility increases with increasing liquid limit, increasing molding water content, decreasing dry density, increasing degree of saturation, and compaction procedures that produce large shearing strains during the compaction process. It is evident, therefore, that the compressibility characteristics of cohesive soils are much more complicated than the compressibility characteristics of cohesionless materials whose behavior is controlled primarily by the relative density and gradation characteristics. Furthermore, the time rate of compression is an important factor in cohesive soils, whereas in cohesionless materials the rate of compression is generally rapid enough to eliminate the consideration of time rate of compression. The influence of these various factors will be discussed in the following sections.

Void Ratio vs Pressure Relationships Between Saturated Undisturbed Cohesive Materials and Compacted Cohesive Materials

The void ratio-pressure relationships of compacted cohesive materials are quite similar to the void ratio vs pressure relationships for undisturbed natural clays providing the sample is not saturated at some intermediate confining pressure. Leonards (36) observed that the compression index decreased for statically compacted clays that were soaked prior to consolidation with a decrease in the as-compacted void ratio. These data do exhibit a rather distinct break in the slope of the void ratio vs logarithm of pressure relationship similar to the change in slope at the preconsolidation pressure observed in undisturbed clays. The pressure at which the change in slope occurred was found to increase with a decrease in the as-compacted void ratio. It may be reasoned that this change in the slope of the void ratio vs logarithm of pressure curve, which is similar to preconsolidation pressure for natural clays, is caused by the built-in soil structure produced by the compaction process. This built-in soil structure is influenced not only by the compaction procedure but also by the ability of the soil to respond to this compaction process. It has been shown previously that the soil structure produced by various compaction procedures is essentially the same for a soil when it is being compacted on the dry side of optimum. However, when the material is being compacted on the wet side of optimum, it has been shown indirectly that the structure would depend on the compaction process. Based on data presented by Seed, Mitchell and Chan (22), it can be seen that the ratio of secant moduli for different compaction procedures varies significantly on the wet side of optimum (Fig. 7). It is, therefore, obvious that the compressibility characteristics for materials that are compacted on the wet side of optimum will be greatly influenced by the compaction procedure used to compact the soil. In general, it appears that the compressibility will increase for a soil compacted by static, vibratory, impact, and kneading compaction methods, in that order. That is, the statically compacted specimens should be less compressible than the specimens that are compacted using kneading methods for the same water content and dry density on the wet side of optimum; however, on the dry side of optimum the compressibility should be approximately the same regardless of the compaction method used.

Yoshimi and Osterberg (37) have presented compressibility data on Vicksburg silty clay prepared by kneading methods. These data also exhibit a distinct break in the slope of the void ratio vs logarithm of pressure relationship similar to saturated undisturbed materials. It is interesting to note from these data that the change in slope that is similar to the preconsolidation pressure occurs at a consolidation stress slightly less than the compaction stress used to prepare the compacted samples. It might be reasoned that the increase in compressibility at values in excess of the compaction pressure is caused by an additional breakdown in the structure of the compacted material once the compaction pressure has been exceeded. Based on this reasoning it may be concluded that if the stresses that will act on a material during its service

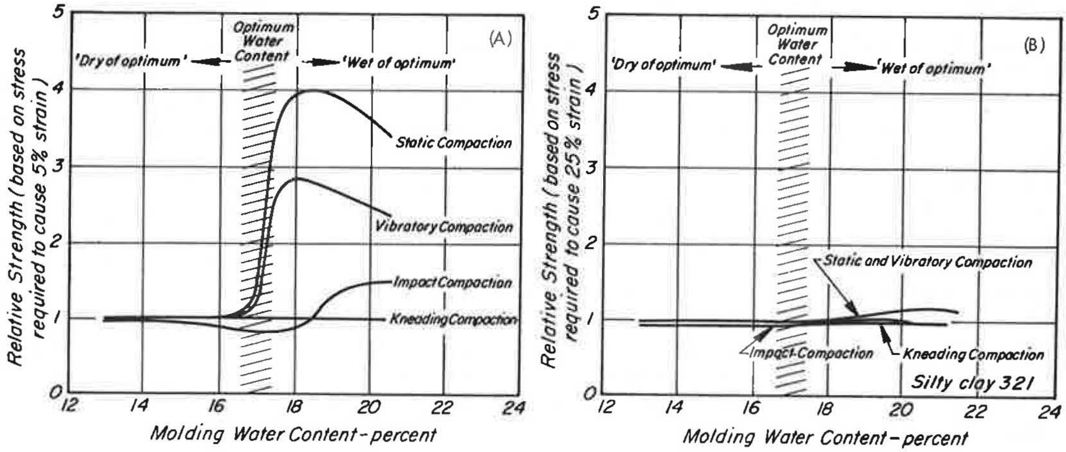

Figure 7. Relative strength at different strain levels for different methods of compaction: (A) performance at 5 percent strain; (B) performance at 25 percent strain (after Seed et al, 1960).

history are lower than the compaction stress, then the compressibility will be minimized; however, if the stresses that will act on the material are in excess of the compaction stress it might be assumed that the compressibility will be much larger.

The reasons for differences in the compressibility characteristics of a material on the wet side and dry side of optimum have been explained by Lambe (26) on the basis of a change in the soil structure that occurs as the material is compacted on the dry side, at optimum, and on the wet side of the compaction curve. These arguments are similar to those proposed by Seed et al based on the secant modulus at low strains. For a fairly small consolidation pressure range it appears that the samples that are compacted wet of optimum will experience a larger change in void ratio than when the material is compacted dry of optimum. However, for a large pressure range it appears that the total change in void ratio or compressibility is essentially independent of the initial conditions. This may be attributed to the fact that at sufficiently large consolidation pressures the soil structure of the material compacted either wet or dry of optimum water content will become highly dispersed and essentially independent of the initial soil structure and, therefore, the overall compressibility will be essentially the same.

Effect of Saturation on Compressibility

The previous section dealt with samples that were tested in the as-compacted state or samples that were saturated prior to testing. In the field, however, the material is generally compacted and then saturation may occur at a later stage when a confining pressure will exist on the material. The effect of saturating the material under various confining pressures was investigated by Jennings and Burland (38). This change in void ratio upon soaking can be quite large and may in fact be of the same order of magnitude of void ratio change that occurs during a large increase in externally applied pressures. For soils that appear to be subject to this collapse phenomenon it appears that increased compaction which produces a decrease in the void ratio will significantly aid in reducing the amount of void ratio decrease that will occur upon saturation of the material. The amount of collapse may also be reduced by increasing the degree of saturation of the material during the compaction process; however, this increase in degree of saturation and/or water content will lead to greater compressibility caused by external pressures.

Correlation of Soil Type With Compressibility

In fully saturated natural soils, it is well known that certain index properties may be used to indicate compressibility characteristics of cohesive materials. For example, the relationship between the liquid limit of a low to medium sensitivity material can be used to estimate the compression index of that material. For compacted cohesive materials, the problem becomes more difficult because not only are the properties of the materials involved but also the effect of the compaction process which is used to compact the soil. Regardless of the compaction procedure, however, certain conclusions can be drawn concerning the relationship between material properties and the compressibility of the material. Investigations by Gould (39) on rolled fill material indicate that the compressibility is significantly influenced by the plasticity of the fines in the soil. It was observed that fine sand and silt with little or no plasticity, when placed dry of optimum, have low compressibility, whereas clays of low to medium plasticity compacted dry of optimum exhibit higher compressibility. It was found that, in general, the compressibility increases in the following order: (a) gravel and sands with silty fines, (b) silts of low plasticity, (c) gravel and sands with slightly plastic fines, (d) sands with clayey fines, (e) mixtures of gravel sands and silts with clay, and (f) clays of low to medium plasticity. Gould concluded that this trend emphasized the importance of the plasticity of the fine fraction on the compressibility compared to gradation or grain size characteristics. Recent laboratory investigations by Matyas (40) provide additional evidence of the fact that compressibility is significantly influenced by the type and amount of fines and also by the molding water content.

It may be concluded that soil type is undoubtedly one of the basic factors influencing the compressibility characteristics of a compacted cohesive material, but additional factors such as the method of compaction, molding water content, and degree of saturation will also have significant effects upon the compressibility characteristics.

CONCLUSIONS

The available data in the literature indicate that dry density alone is not always a reliable index of shear strength and compressibility of compacted materials. Several other factors also play an important part in determining engineering properties of these materials. The following conclusions may be drawn from a review of the literature on shear strength and compressibility of compacted materials.

1. The shearing resistance of compacted cohesionless materials is related to properties of the material and the density obtained by compaction. The most important factors that will produce increasing shearing resistance are increasing angularity of the particles, increasing surface roughness and improved gradation. Improved gradation and possibly increasing amounts of larger-grained material mainly increase the amount of dilation during shear, which leads to increasing shearing resistance.

For a given cohesionless material the shear strength is inversely related to the void ratio or directly related to the dry density obtained by compaction. This relationship is valid regardless of the compaction method used and any strain up to peak strength.

2. The compressibility of a compacted cohesionless material is influenced by the same factors that influence the shear strength. In general, the compressibility decreases with improved gradation and decreasing as-compacted void ratio. Unlike the effect on shear strength, increasing angularity will produce increasing compressibility.

3. Based on effective stress theory, it can be shown that the initial effective stress may either increase or decrease with increasing water content along a compaction curve on the dry side of optimum, but that the effective stress will always decrease with increasing water content along the compaction curve on the wet side of optimum.

4. Cohesive soils are found to have differences in shear strength that are caused by differences in soil structure. A flocculated soil structure is more rigid and produces smaller initial pore water pressures during shear than the same soil with a dispersed soil structure. This leads to increased strengths, particularily at low strains. The soil structure that is produced by compaction is governed by the soil type, the molding water content, and the compaction method.

5. The as-compacted shear strength of a cohesive soil for a constant dry density will always exhibit a decrease in shear strength with an increase in water content. In fact, most data indicate that the as-compacted shear strength will decrease over the entire range of water contents usually investigated, even though there is an increase in dry density with an increase in water content on the dry side of optimum water content.

6. The as-compacted shear strength of a cohesive soil, for a constant water content, will exhibit an increase in shear strength for all water contents with an increase in dry density only when the strength is defined at large strains. At low strain levels the strength may increase or decrease with dry density depending on the water content and the method of compaction.

7. For soaked conditions, the resulting shear strength is determined by the combined effect of swelling during soaking, initial water content, and as-compacted soil structure. For CBR-type tests that allow swelling to take place it appears that the maximum soaked shear strength occurs at approximately the as-compacted optimum water content.

8. The strength of a compacted cohesive soil may change significantly with time after compaction because of thixotropic effects.

9. Compressibility of compacted cohesive materials is influenced by soil type, molding water content, as-compacted dry density, initial degree of saturation and compaction method.

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Evaluation of Soil Compaction by Rheological Techniques

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The object of this study was to determine what basic characteristics of soils must be used to better specify desired soil compaction. The densification of soils was studied from a rheological point of view, using data from laboratory tests. The plan of the study was to describe the mechanical properties of highway subgrade materials by fundamental strength properties, which can be used to show changes in the strength properties of the material caused by the type or amount of compactive energy applied.

Experiments were performed to validate the application of the linear viscoelastic theory and mechanistic models to soils, and to determine the limitations of such approaches to compaction problems. Various types and amounts of compaction energy were programmed for the selected soils. Experiments were conducted using unsaturated soils over a range of molding water contents, input of compaction energies, saturations, dry unit weights, stress-strain levels and other environmental conditions.

Stress relaxation experiments as well as confined and unconfined constant-load creep tests were carried out to study the mechanical response of soils on the phenomenological level prepared by several different types of compaction methods. The study utilized the electrical-mechanical analogy and the complex elastic modulus to define the mechanical response of soils.

•IN HIGHWAY and airfield construction projects, soils are used as embankment materials, and as a result are required to support both the static load of the overburden pavement system and the transient traffic loads under all environmental conditions. To place the soil in the optimum state so that the material is able to support these loads under all adverse conditions, the soil is mechanically stabilized by the process of compaction. Compaction is generally defined as densification of soil by the application of external mechanical energy to improve the strength, to increase the stiffness, to reduce permeability and swelling characteristics, and to improve other properties of the soil for better overall performance under service conditions. The principal variables which influence the state of the compacted soils are the type and amount of compaction energy, soil texture, and moisture content (1, 2, 3, 4, 26, 27).

To calculate the stress and the deformation experienced by the subgrade, the stress-strain characteristics under normal working conditions of the compacted soil are required as well as the mechanical properties of the other components of the pavement system. Generally, a study is performed in the laboratory utilizing soil specimens compacted by laboratory compaction methods which simulate field construction, although direct field compaction studies would be more useful. For a given compaction

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energy the soil specimens compacted at the optimum moisture content yield the maximum dry density, and these test specimens are generally employed in determining the strength and deformation characteristics of the soil. In many cases the design strength of the soil is based on empirical strength values taken from test specimens at their worst anticipated condition in the field, obtained by soaking the material. However, if proper drainage is achieved, this process is questionable.

In order to evaluate the effectiveness of compaction in the field construction, the general practice is to utilize the dry unit weight of the compacted soil as a parameter which in turn is obtained by using nuclear devices, sand cone, balloon, etc. Other parameters in use today are the needle penetration resistance, seismic, bearing, shear strength evaluated by vane shear tests and the unconfined compressive strength when final strength is specified. These parameters are not a direct measure of the mechanical response of soil as a subgrade, but they indirectly indicate the strength of the material. It is well known that the strength-deformation characteristic of a soil at or near the failure state is different from that at low stress-strain states under in-service conditions. A more rational method for evaluating compaction characteristics may be a technique in which the soil is subjected to the stresses and environmental conditions similar to those it will be subjected to under in-service conditions, and then the fundamental strength properties of the material are evaluated. In some pavement design theories, the materials constituting the several load-distributing layers are treated as ideal elastic materials in order to approximately determine the stresses and strains in different layers (5, 6). It is well known that no pavement material is perfectly elastic because all materials have time-dependent stress-strain characteristics. Therefore, in defining the stress-strain time-dependent rheological characteristics of a compacted subgrade. the compacted soil may be more rationally treated as a linear viscoelastic material under specified conditions. Soil, being a natural material, is not an ideally linear viscoelastic material, just as there is no perfectly elastic Newtonian or plastic material (7, 8). But within a certain stress-strain range comparable to the highway service conditions measured at the AASHO Road Test, the mechanical behavior of soils, as an engineering approximation, can be treated like that of a linear viscoelastic material. The AASHO Road Test field measurements indicate that the stresses and strains experienced by the subgrade are low when compared to the loads and deformations at the failure state as measured by the unconfined compressive strength. The low stress-strain state experienced by subgrades has been shown by this research to be within the linear range of the soils and hence, as an engineering approximation, compacted soils used for the subgrade of pavement structures can be treated as linear viscoelastic materials. By establishing the linear viscoelastic nature of soils, the viscoelastic constants of the material can be evaluated by any of the rheological tests which are all interrelated by the viscoelastic theory (9, 10).

The objective of this study was to investigate the possibility of evaluating soil compaction utilizing rheological techniques and the limitations of such approaches. This was accomplished by evaluating the rheological strength parameters of compacted soils which are fundamental properties of the material and independent of the type of test. These parameters have been utilized to evaluate the optimum combination of (a) the amount of compaction energy utilized, (b) the type of compaction energy applied, and (c) the molding moisture content for the soils used in laboratory tests. Once having established that the viscoelastic procedure can be used to more rationally evaluate the state of a compacted soil and supplement present techniques, then evaluation for a given soil can be better accomplished under many climatic and loading conditions. The parameters used to evaluate soil compaction also have immediate applications in pavement design techniques (11, 12), which utilize the fundamental strength properties of the material, the complex moduli (13).

MATERIALS

The following four soils were rigorously investigated in this study.

1. Kaolin Clay—This plastic kaolinite clay mined and processed in Edgar, Florida, is a relatively uniform soil when compared to other natural soils. The major portion

TABLE 1

CLASSIFICATION OF THE FOUR SOILS INVESTIGATED

Soil	Atterberg Limits			Specific Gravity	Soil Classification		
	LL (%)	PL (%)	PI (%)		Unified	AASHO	FAA
Kaolinite clay	58	36	22	2.60	MH	A-7-5 (16)	E-8
IITRI clay	37	15	22	2.70	CL	A-6 (13)	E-7
Clayey sand	20	19	1	2.70	SC	A-2-4 (0)	E-3
Silty clay	25	18	7	2.72	CL	A-4[a] (8)	E-6

[a]Ohio specification (A-4b).

of the research effort was concentrated on this soil because of its relatively uniform composition and negligible thixotropic effects.

2. IITRI Clay—A moderately plastic clay used by IITRI in the field study phase of this research.

3. Silt—A natural soil obtained in the Cleveland, Ohio, area.

4. Clayey Sand—Another natural soil obtained at a construction site in Columbus, Ohio.

Considering the nonuniformity of almost all of the natural soils, it was decided to use the relatively "ideal" mined soil, Kaolin, to perform the major portion of the basic research work reported here and to obtain the trend of the soil's response. It was postulated that the research results observed for Kaolin would be applicable to other natural soils. This was found to be true after subsequent tests were performed on the other three natural soils to confirm the research results. ASTM standard tests for identification and classification of the materials were conducted on the four soils (Table 1). In this paper only the results of the research on the kaolinite soil are presented for the impact compaction method. Comparable results and trends were obtained for the other three soils and the kneading type of compaction investigated.

Sample Preparation

The soil passing a number 12 sieve was oven-dried and mixed with the required amount of distilled water following the mixing procedure specified in ASTM D 698-64T. The mixed soil was then stored in a sealed plastic container in a humid room for a minimum of 24 hr to allow the molding water to equally distribute itself within the soil.

Test specimens were prepared using the drop hammer (impact) type of compaction. The mechanical drop hammer compaction apparatus developed and built at Ohio State University is described in detail elsewhere (14). The input compaction energy was varied by changing the total number of drops or blows applied to the sample. The human element involved in preparing the soil samples is critical and extreme care must be used by the laboratory technician to obtain identical samples.

The mold used produced a sample of 1.3125 in. diameter and 2.816 in. in height after the sample was trimmed and extruded. All samples were prepared by compacting the soil into the mold in five equal layers. After extruding the sample out of the mold, the test specimen was weighed, wrapped in a plastic bag, and completely coated with wax. The samples were then stored in a humid room until testing and also until all thixotropic strength changes in the soil with age were negligible. Each data point was reproduced by repeating each test a minimum of four times.

TESTING APPARATUS AND EXPERIMENTAL PROCEDURE

To evaluate the rheological parameters for a linear viscoelastic material the following basic rheological tests were conducted: (a) confined and unconfined creep tests, (b) stress relaxation tests, and (c) deformation at a constant rate of strain.

Creep Tests

The creep test apparatus consists of a Clockhouse triaxial cell, two linear displacement transducers (LVDT), one for measuring the axial deformation and the other for measuring lateral deformation. The transducers are connected to a two-channel Brush recorder or a Sanborn recorder to obtain continuous recordings of the axial and lateral deformations. Samples were enclosed in two rubber membranes, and after the rheologic test, the samples were tested for unconfined compressive strength and evaluation of moisture content. The creep test procedure generally consisted of cycling the desired load through two load and unload cycles, each of 5 min duration, in order to condition the sample under the specified load. The third loading cycle which was used to obtain the experimental data was of 15 min duration in the case of short-term creep tests and approximately 1500 min duration under load in the case of long-term creep tests and 15 min in the unload state. Axial strain was measured with an LVDT and a dial gage in order to check the deformation readings.

Stress Relaxation Experiments

The stress relaxation apparatus consisted of a Genor triaxial cell and Instron testing machine. The stress generated was measured by a load cell and continuously recorded. The relaxation tests basically consisted of applying a predetermined constant strain and measuring the stress generated as a function of time. The test procedure consisted of applying a predetermined deformation and allowing the stress relaxation to take place for a 10-min period. Then the applied deformation was removed until the stress generated was zero. This straining and relaxation cycle was repeated seven times. The stress response in the sixth cycle was used to obtain the data.

Unconfined Compression Tests

In unconfined compression tests, a relation between stress and strain is obtained by continuously axially straining the sample at a constant rate of strain until failure. The test was performed following ASTM specification D 2166-63T using a Karol Warner unconfined compression tester at a strain rate of 2.82 percent per min.

RELATED THEORY AND EXPERIMENTAL RESULTS

Notation

The following notation is used throughout the ensuing discussion:

$E_c(t)$ = creep modulus;
$E_r(t)$ = stress relaxation modulus;
$|E^*|$ = absolute value of complex elastic modulus;
V^* = complex Poisson's ratio;
t = time;
ϵ = strain;
$\epsilon(t)$ = time-dependent strain;
γd = dry unit weight;
S = saturation;
σ'_c = ultimate unconfined compressive strength;
η_o = viscous coefficient of Newtonian flow;
σ_{zz} = axial stress;
ϕ_E = phase angle of E^*;
ω = circular frequency in radians per second;

W = water content; and

λ_o = coefficient of tractive viscosity.

Dry Density—Moisture Content—Compaction Energy

The principal variables investigated utilizing the Kaolin soil were (a) the molding moisture content, (b) soil saturation, (c) the amount of compaction energy and (d) the type of compaction energy applied. The five molding water contents ranged from 21 to 31 percent, which covered both the dry and the wet side of the optimum moisture content for the compaction energy levels studied. The degree of saturation ranged from 95 to 65 percent.

Using the mechanical drop hammer device, it was possible to prepare identical samples with normal care by technicians, and this device expedited the research work on the three other natural soils. The five levels of compaction energy used were 25, 40, 60, 80, and 120 total blows per sample. Figures 1 and 2 show dry density and unconfined compressive strength vs moisture content. After extruding the compacted soil samples from the mold the specimens were weighed and representative samples were tested for unconfined compressive strength, σ'_c, and the moisture contents determined.

Thixotropic Tests

In order to study the thixotropic characteristics of the compacted soil, identical soil specimens were prepared and placed in plastic bags which were coated with wax and then stored in a humid room until tested. Specimens were tested in unconfined failure strength experiments and in constant-load creep tests at different ages to study the effect of age on the strength characteristics of the compacted soil. The unconfined compressive strength, σ'_c, and the axial strain, ϵ_{zz}, at a loading time of 30 sec due to an axial stress of 12 psi in the third loading cycle, were used as the strength parameters to determine the age at which the strength of the remolded compacted soil reached an equilibrium state. Tests indicated that Kaolin soil has no significant thixotropic characteristics and the curing age has a negligible effect on the creep and failure strength of the soil.

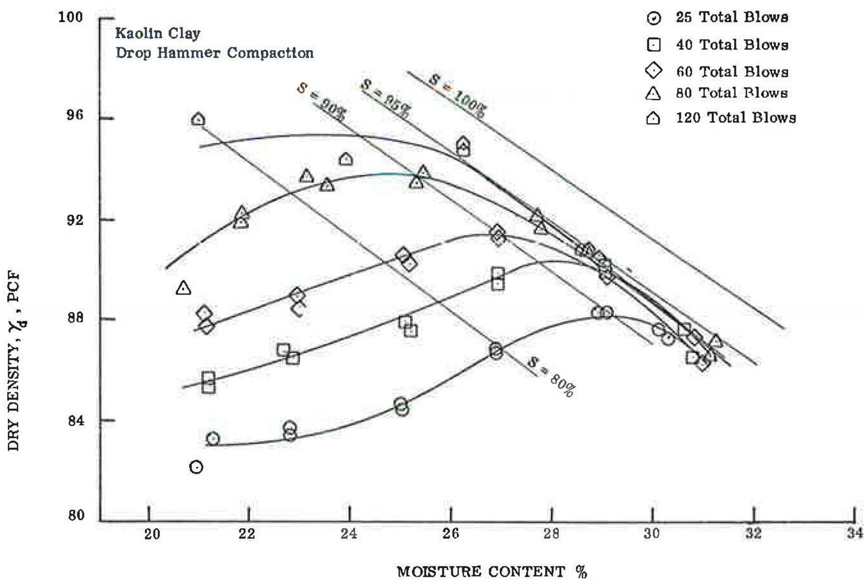


Figure 1. Dry density vs moisture content.

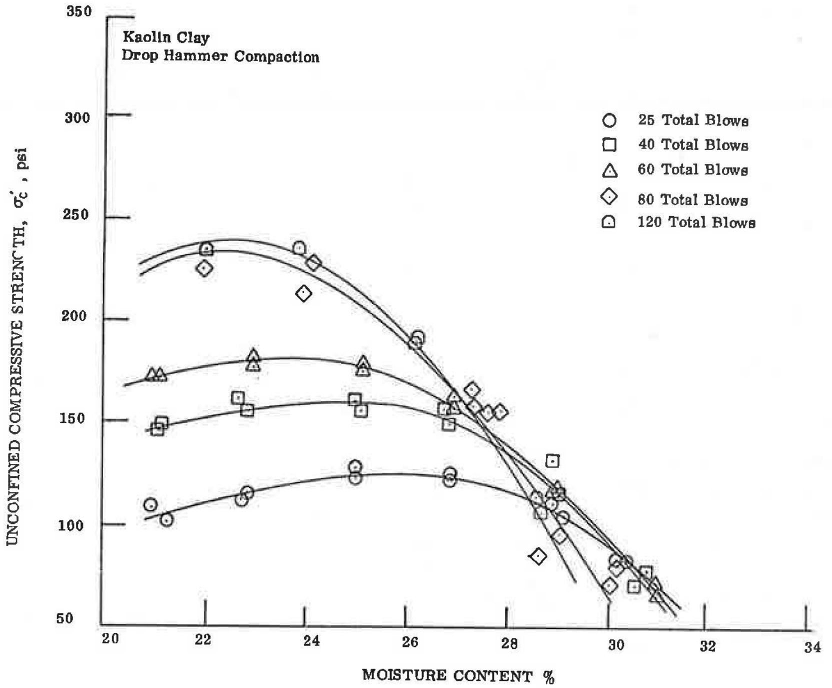


Figure 2. Unconfined compressive strength vs moisture content.

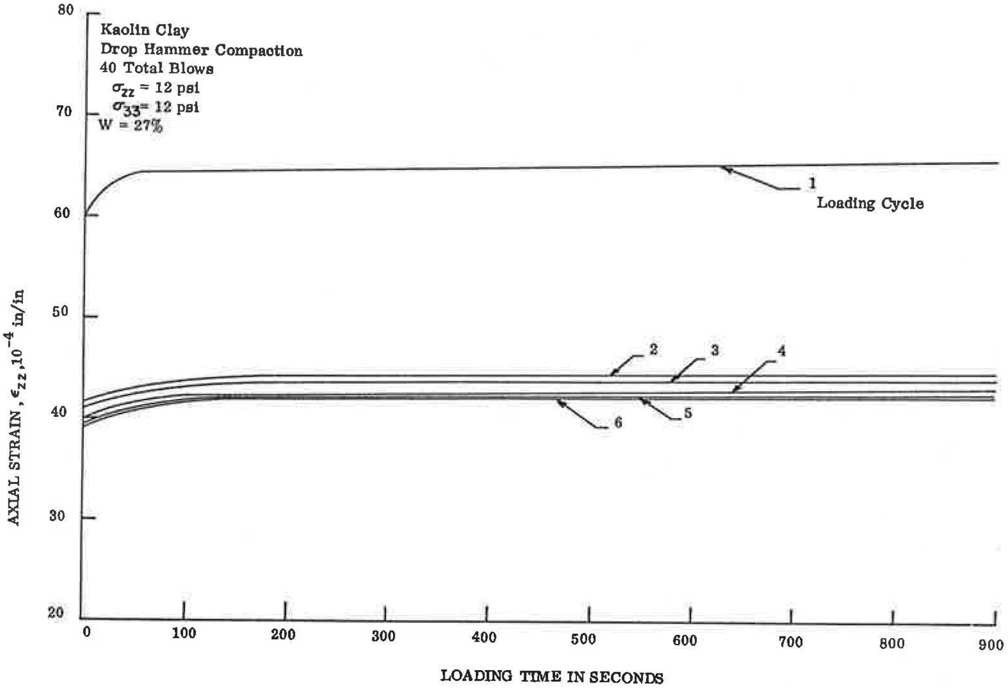


Figure 3. Relative axial strain vs time for six loading cycles.

Mechanical Conditioning of Soil

Constant-load creep tests in both the confined and the unconfined state as well as stress relaxation tests were the principal rheological tests performed in this study. It was found that the axial and lateral strain response of the soil under load in the first loading cycle was significantly higher than in the subsequent loading cycles. This phenomenon may be due, among other factors, to the initial seating adjustment of the LVDT and soil sample (even though a small set load is used in the loading procedure), a decrease in the void ratio because of decrease in the air content of the soil, and a form of mechanical conditioning of the soil under load. The load to apply an axial stress of 12 psi was cycled six times, each cycle consisting of 15 min under load and 15 min under no load. The relative axial strain in each load cycle and relative axial recovery strain during the unloading period were plotted vs time. Figure 3 indicates that the second cycle and all later cycles are approximately constant. Therefore, all the creep test data were obtained from the response of the material in the third load cycle.

It was also found that reducing the duration of the mechanical conditioning cycle from 30 to 10 min during the first two cycles had a negligible effect on the response of the material in the third cycle. Therefore, a load cycling pattern of 5 min under load and 5 min under no load was used for the first two cycles. The third cycle, which was used to obtain data, was either of 15 min duration under load in the short-term creep tests or up to 1500 to 2000 min duration under load in the long-term tests when the soil was allowed to reach a steady-state strain condition. In each case the rebound of the soil was evaluated for 15 min or greater.

Stress Relaxation Tests

In the stress relaxation test, a constant strain is applied to the specimen and the stress generated as a function of time is evaluated. After a given time the strain is removed until the specimen relaxes to the zero stress state. The response under the first cycle of strain is significantly different, as in the creep tests, from the response of the soil in subsequent cycles. The stress relaxation modulus, $E_r(t)$, which is defined as the time-dependent stress at any time, t, divided by the applied constant strain, was utilized as a rheological strength parameter to study the response of soils under constant strain cycles. In the cases where the moisture contents are dry of optimum, the samples seem to reach a stable condition after the second or third cycle, while the samples with a moisture content higher than the optimum did not appear to reach a steady-state condition until the sixth cycle. Therefore, the response under the sixth cycle was used to evaluate the rheological strength and deformation properties of the soil.

Viscoelastic Linearity Experiments

The viscoelastic linearity tests, which were extremely important to this phase of the study, were performed to determine if materials such as unsaturated soils comparable to those used in highway embankments can be defined by linear viscoelastic concepts, and to determine the range of stresses and strains within which these principles are applicable to soils. The linear viscoelastic response of a material is, of course, a function of the environmental conditions of the test and the material studied. Similar restrictions are also required in the classical elastic, plastic and Newtonian liquid theories. The axial, radial, and volumetric strains at several loading times in the third loading cycle were used as parameters to evaluate the linear viscoelastic range of stress in constant-stress creep tests in both the confined and the unconfined test conditions. Identical specimens were tested in the constant-load creep tests in which the axial stress varied from 4 psi to 120 psi. The load in each test was cycled as previously mentioned, and the strains at selected loading times were analyzed. In the case of drop hammer compacted specimens, the linear viscoelastic range (15, 16, 17) for the unconfined state was up to 20 psi, as shown in Figure 4. In the confined creep tests with a confining pressure of 12 psi, the linear range of axial stress was

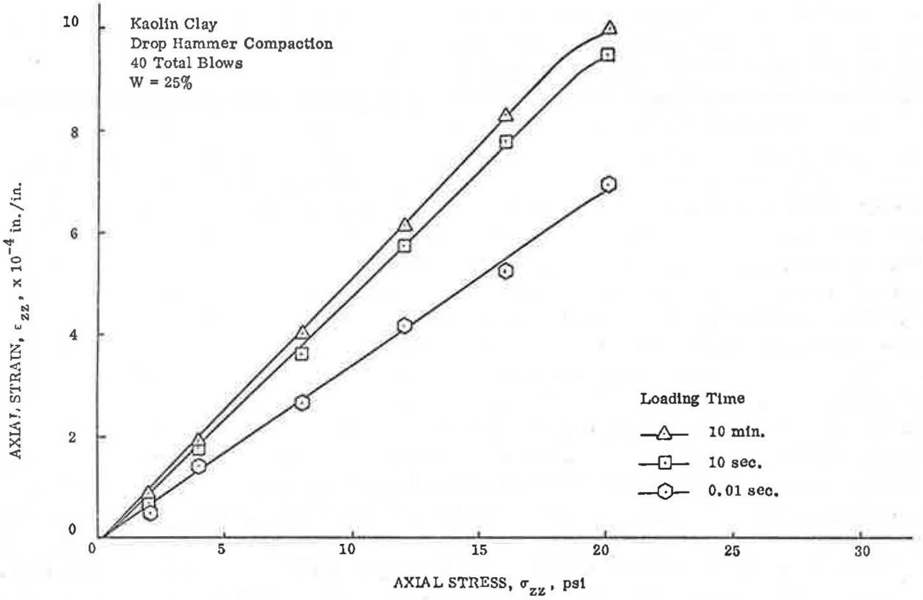


Figure 4. Axial strain vs axial stress at three loading times.

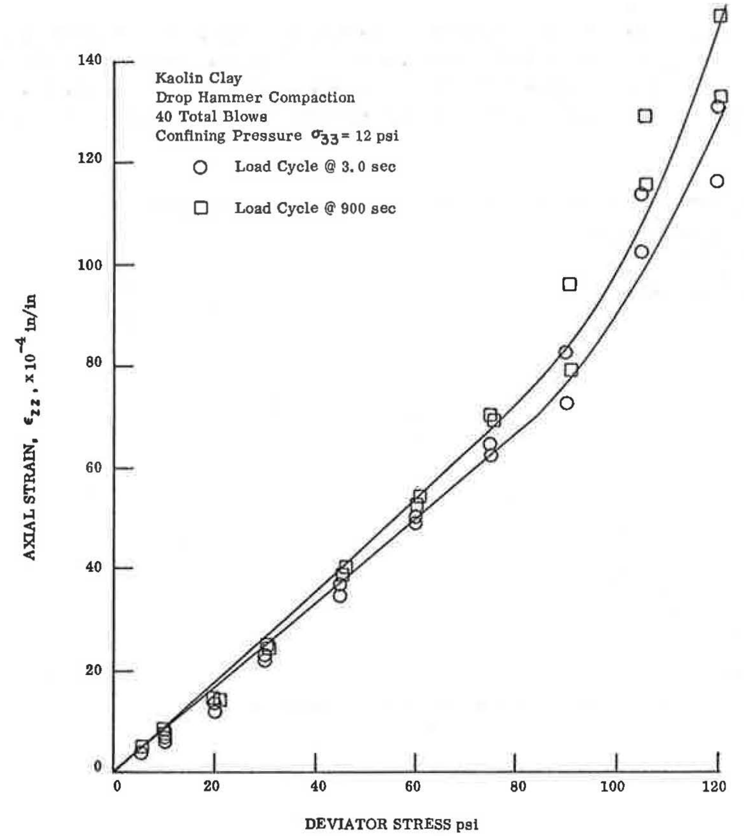


Figure 5. Axial strain vs deviator stress at two loading times.

increased to 75 psi. Figure 5 shows the triaxial creep linearity tests; the straight line portion of the graph indicates the linear viscoelastic range of stress. An identical procedure was used to evaluate the radial and volumetric strains from the same constant-load tests performed on the Kaolin. The radial, the axial and the volumetric strains illustrated that the linear viscoelastic theory is a good approximation to depict the response of the soil at the stress levels and conditions studied. The linearity data were obtained from the strain-time plots in which only the stress level is varied.

Another form of rheologic test used in this study to evaluate the linear viscoelastic response of soils was the stress relaxation test. Identical specimens were subjected to different strains ranging from 0.1 percent to 0.84 percent, and the stress generated at times of 0, 24 and 600 sec in the second and sixth straining cycles were selected as parameters to determine the linear range. Figure 6 shows stress vs strain obtained from the relaxation tests; the straight line portion indicates the linear viscoelastic range. The drop hammer compacted Kaolin soil is found to be linear in the stress relaxation tests up to a strain of 0.7 percent, generating a stress of nearly 100 psi. Of course, this linear range is a function of many factors, such as soil type, soil structure, molding moisture content and the testing environmental conditions. Therefore, within the linear range the response of compacted soils to stress and strain can be studied from the material science point of view as a linear viscoelastic material.

The classical elastic theory deals with the response of purely elastic materials where stress is proportional to strain. However, elastic materials are idealizations. Real materials existing in nature generally show stress and time anomalies. A viscoelastic material is one which exhibits both elastic and viscous characteristics, and stress is related to strain by a function of time. To describe the response of such materials by the model representation (13, 17), a mechanical system is used consisting of Hookean springs and Newtonian dashpots connected in series or parallel in various configurations.

The mechanical response of materials may be approximated by models composed of a finite number of springs and dashpots such as the Kelvin, Maxwell and Burgers models (15, 18), which have been applied by many engineers in studying polymers. Several authors (19, 20, 11, 29) have suggested that it is possible to represent the response

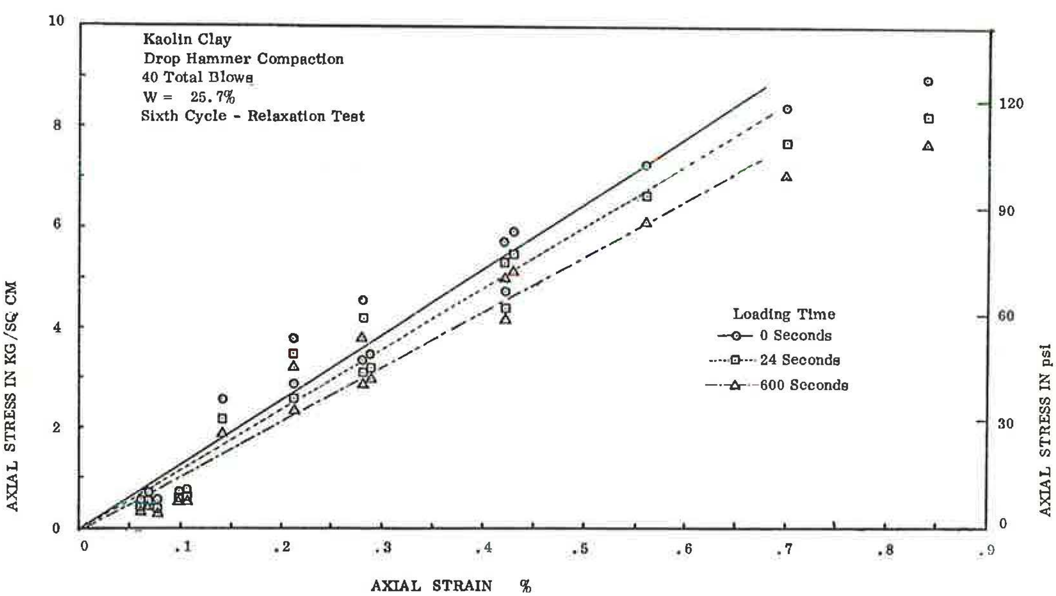


Figure 6. Axial stress vs axial strain.

of viscoelastic materials using more refined models consisting of a larger number of elements in the model. Such concepts have been applied to bituminous materials with success.

The generalized Voigt and Maxwell models consist of n + 1 and n number of elastic and viscous elements, respectively. Under a constant stress the Voigt model exhibits creep behavior; the creep compliance, Jc (t), used in this study is defined as the time-dependent axial strain divided by the constant axial stress:

$$J_c(t) = \frac{1}{E_o} + J_\psi(t) + \frac{t}{\eta_o}$$

where

$$J_\psi(t) = \sum_{i=1}^{n} J_i(1 - e^{-t/\tau_i})$$

Such a model under given conditions has constant element parameters defining the material properties. The generalized Maxwell model represents a continuous spectrum of relaxation times. In a parallel arrangement the stresses are additive and response of the material is described by

$$E_r(t) = \sum_{i=1}^{n} E_i\, e^{-t/\tau_i}$$

Each Maxwell element consists of a spring and dashpot in series. In addition to the model theory there are several other methods for specifying the response of a linear viscoelastic material such as the operator equation and the electrical analogy (21, 22).

The general stress-strain equations of a linear viscoelastic material are defined in the frequency domain in terms of algebraic coefficients which are only functions of frequency. These coefficients are complex numbers whose magnitude and phase define the properties of the material. The detailed analytical and graphical methods for obtaining the complex moduli of a given material are presented in the literature (11, 21, 28, 29, 30). The parameters may be obtained experimentally by a series of dynamic tests, a single static creep test or a stress relaxation test. The first method yields discreet pairs of the magnitude and phase of the moduli at each frequency used, while the second and third methods yield analytical expressions for the modulus as a continuous function of frequency.

The static test methods were applied to determine the complex creep modulus, E*, and the complex transverse modulus, T*, of soils. The phenomenological theory of linear viscoelastic behavior is of great value for interrelating the dynamic, creep and stress relaxation types of experimental measurements, and for describing the response of soils in the time or frequency domain. In theory, once the response of a linear viscoelastic material is evaluated in one type of test (creep), the response of the material in different independent types of tests may be determined (dynamic and stress relaxation) (13). The literature shows that, by means of the linear viscoelastic theory, it is possible to represent the response of a viscoelastic material with a mechanical model and predict the response of the material in the other types of tests (18). Both creep and stress relaxation tests were performed and an excellent correlation between the results of the two types of tests was obtained in this study.

Using the interrelations among the viscoelastic materials, the complex moduli of the material were determined from the static test results. The impedance of the mechanical system, E*, using a Voigt model representation can be shown to be of the form

$$E^*(j\omega) = \frac{1}{\frac{1}{E_1 + j\omega\lambda_1} + \frac{1}{E_2 + j\omega\lambda_2} + \cdots + \frac{1}{E_n + j\omega\lambda_n} + \frac{1}{j\omega\lambda_o} + \frac{1}{E_o}}$$

where E and λ are the spring and dashpot constants, respectively (11).

As shown by the several types of linearity tests, the compacted Kaolin soil can be approximated to a linear viscoelastic material within the normal stress ranges experienced in highway embankments (10, 24). For a linear viscoelastic material it is possible, in theory, to transform the creep compliance directly to the stress relaxation modulus. The mathematical relations and formulas relating the creep compliance to the relaxation modulus have been developed by Secor and Monismith (20) and Ferry (18). The References contain the complete details of the mathematical development of the necessary equations. The equation used for this transformation of creep compliance, $J_c(t)$, to the relaxation modulus, $E_r(t)$, is of the form

$$E_r\left(\frac{t_{n-1}}{2}\right) = \frac{t_n - \sum_{i=o}^{n-2} E_r\left(t_{i+\frac{1}{2}}\right)\left[f\left(t_n - t_i\right) - f\left(t_n - t_{i+1}\right)\right]}{f\left(t_n - t_{n-1}\right)}$$

where

t = time,

$E_r(t)$ = stress relaxation modulus,

$f(t)$ = $\int_o^t J_c(\tau)\, d\tau$, and

$J_c(t)$ = creep compliance.

By the use of the above equation and the assumption that in the numerical integration the creep compliance is assumed to be constant between $t = 0$ and $t = t_1$, the first time increment, this assumption allows the first value of the compliance function to be defined by

$$E_{1/2} = \frac{t_1}{f_{(t_1)}}$$

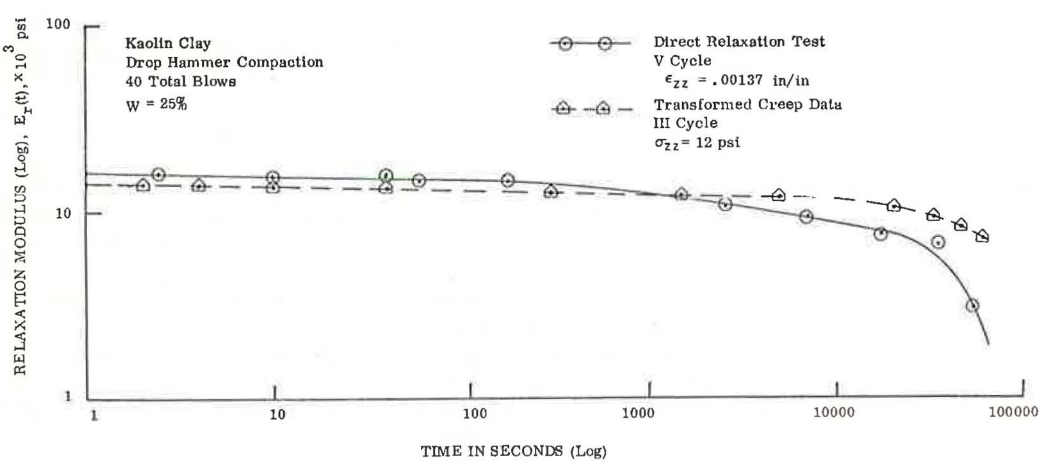


Figure 7. Relaxation modulus vs time.

If we are dealing with a linear viscoelastic material it would then be possible to calculate the values of the stress relaxation modulus over the same time range as those obtained from the creep test. An IBM 7094 Scatran computer program was written to transform the creep compliance function to the relaxation modulus function.

Figure 7 shows stress relaxation modulus vs time obtained directly from stress relaxation tests and also by the transformation procedure mentioned using creep test data. The excellent agreement of the data can be noted. The test specimens were conditioned identically for these tests by use of the Instron testing machine. The close agreement between the two tests is a direct measure of the degree to which the compacted soil is a linear viscoelastic material, and provides additional verification of applying rheological concepts to soils.

EVALUATION OF COMPACTION BY RHEOLOGICAL TECHNIQUES

Rheological parameters of a linear viscoelastic material can be evaluated from any or all of the rheological tests previously mentioned at stresses within the linear range. Data from the third loading creep cycle, such as the axial and radial deformations, can be evaluated and the instantaneous deformation determined at very low loading times, down to approximately 0.01 sec.

The complex elastic modulus, E*, is a complex number consisting of a real part, $E_1(\omega)$, which is made up of the instantaneous elastic response as well as portions of

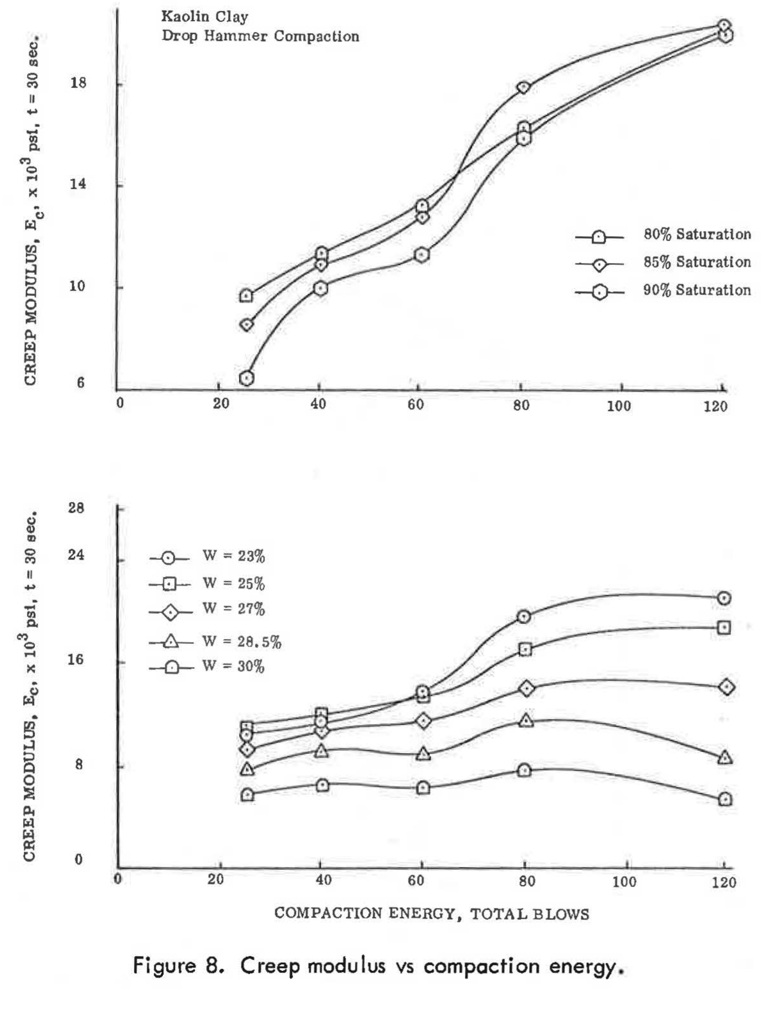


Figure 8. Creep modulus vs compaction energy.

the retarded elastic response, and an imaginary part, $E_2(\omega)$, which includes a part of the time-dependent elastic component and the total viscous response of the soil under load (23, 18). It should be noted that in many cases the axial deformation at 30 sec may be roughly equal to the instantaneous elastic response. The elastic portion of the creep modulus, E_c, at a loading time of 30 sec was also used as a rheological parameter to study the effect of the principal variables of moisture content and the input of compaction energy. The axial strain data as functions of time were used to evaluate the absolute value of the complex modulus, $|E^*|$, and the phase angle of the modulus, ϕ_E, as functions of loading frequency by transforming the experimental results from time domain to frequency domain.

Soil Specimens Prepared by Impact Compaction

Using samples prepared by the drop hammer type of compaction energy, creep tests under an axial stress of 12 psi were performed. The elastic creep modulus evaluated at a loading time of 30 sec was used as a parameter for the analyses of the data. In Figure 8, plots of the elastic creep modulus vs compaction energy are presented for constant molding moisture contents and degrees of saturation. Similar plots to evaluate compaction using the magnitude and phase of the complex modulus, E*, as parameters are shown in Figure 9.

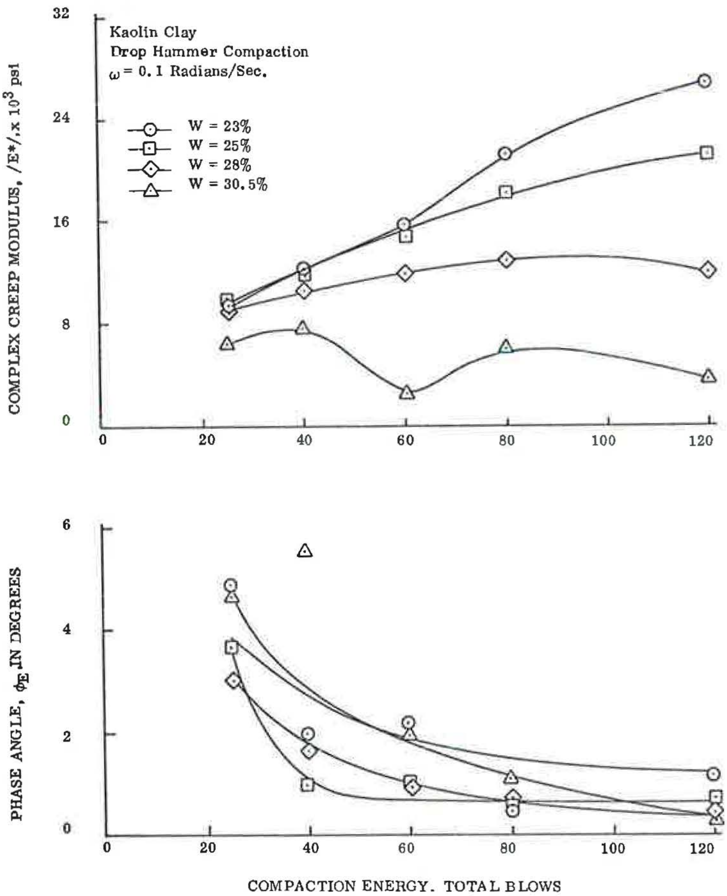


Figure 9. Complex creep modulus and phase angle vs compaction energy.

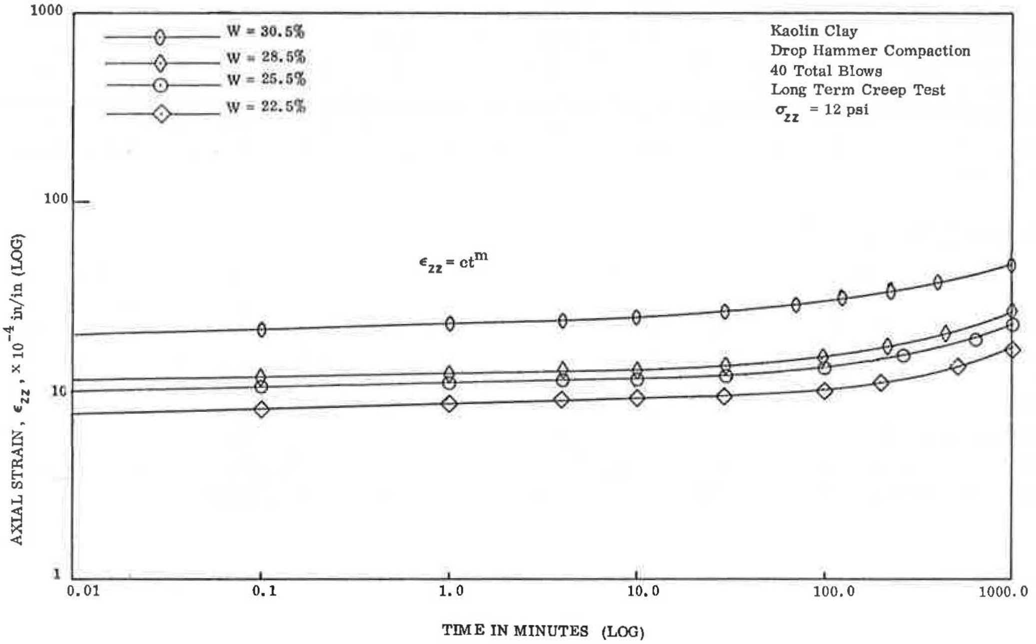


Figure 10. Axial strain vs time in creep test.

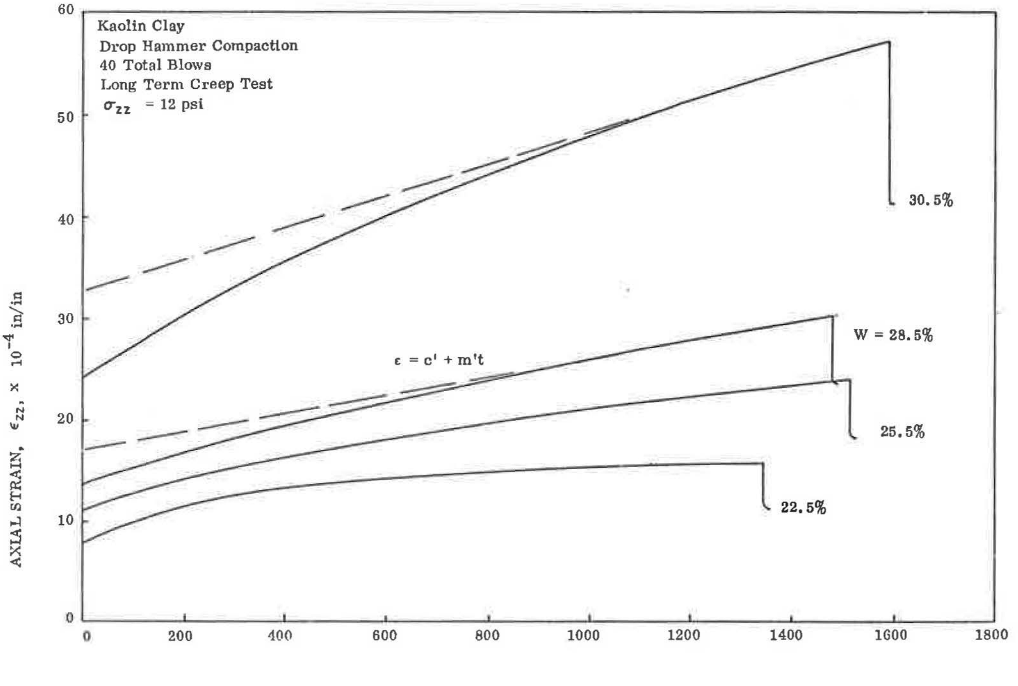


Figure 11. Axial strain vs time.

Curve fitting techniques were applied to the typical plots of the basic creep test data of strain vs loading time to fit an exponential equation to the strain-time creep curve using a method described in detail in the pavement design literature (11). A second Scatran computer program was written to transform the exponential strain-time equation from the time domain to the frequency domain and determine the magnitude and phase of E* and T* over a wide range of frequencies from 0.1 to 1000 radians per sec.

Long-Term Creep Tests

A second type of constant-load creep test was utilized to study the long-term response of the soil in which the specimen was under load until the strain reached a linear steady-state flow condition. Figure 10 shows the axial strain vs time on a log-log scale. It should be noted that strain and time are related by a power function of the form $\epsilon = ct^m$ in the initial stages of the mechanical response of the soil, as indicated by the straight line portion on the plot. Strain and time also related by an equation of the form $\epsilon = c' + m't$, which is the linear relation indicated by the straight line portion of the curve on the natural scale plot shown in Figure 11 during the steady-state portion of creep behavior. The slope of the steady-state portion or steady-state

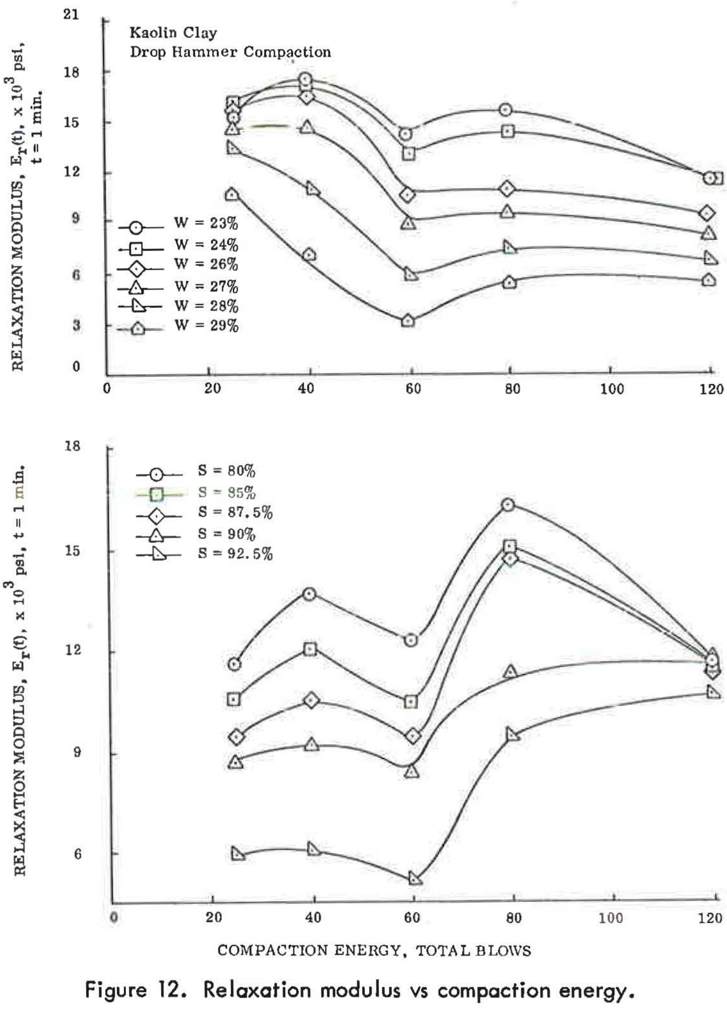


Figure 12. Relaxation modulus vs compaction energy.

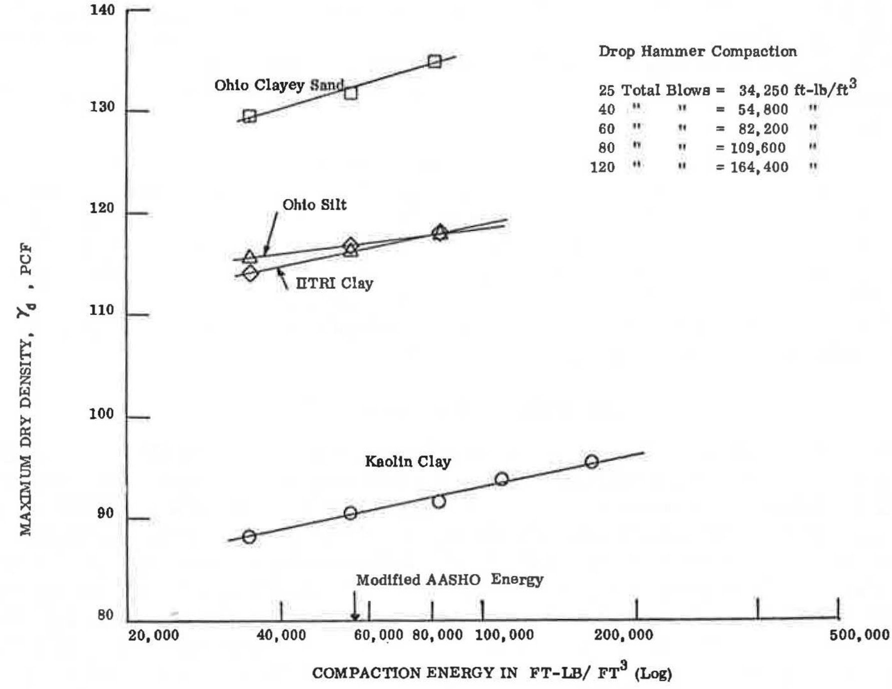


Figure 13. Maximum dry density vs compaction energy.

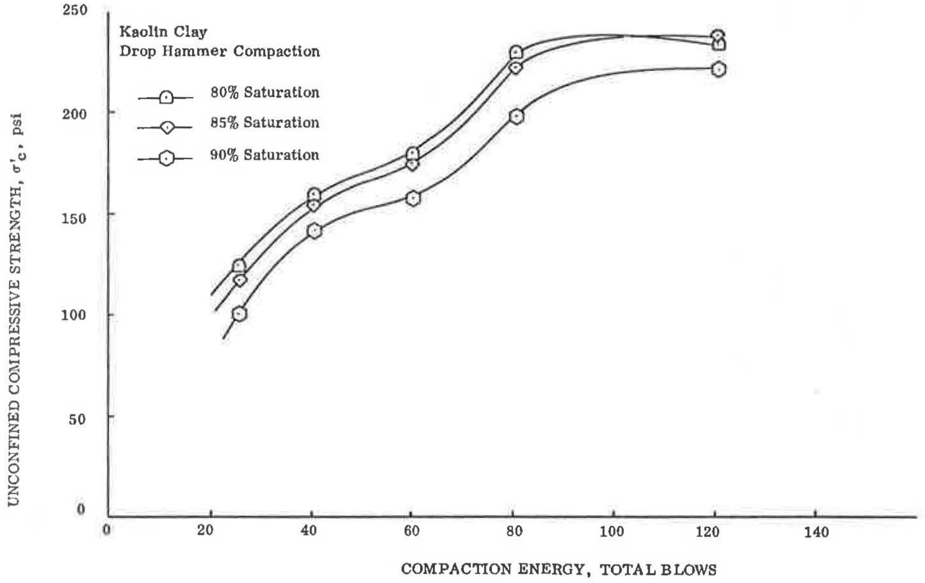


Figure 14. Unconfined compressive strength vs compaction energy.

strain rate during the final stages of the creep plots can be used to find the coefficient of viscosity, λ_o, of the soil. This λ_o represents the viscosity element of the outside dashpot if the material is represented by a generalized Voigt model.

Stress Relaxation Tests

Another independent basic rheological test performed in this study was the classical stress relaxation test. At any experimental time it was possible to calculate the stress relaxation modulus, $E_r(t)$, which is defined as the time-dependent stress at a given time divided by the applied constant strain. Because most of the relaxation of stress takes place within the first minute, the $E_r(t)$ at 1 min was selected as a parameter to study the effect of the variables of moisture content and input of compaction energy. Figure 12 shows interpolated data of the relaxation modulus $E_r(t)$ vs compaction energy at constant moisture contents and also constant saturations.

DISCUSSION OF RESULTS

The effect of compaction energy on the selected rheological parameter can be studied either at a constant moisture content or at a constant degree of saturation. By interpolation of the basic experimental data, it is possible to plot the soil parameters of dry density, unconfined compressive strength, creep modulus, complex moduli, and others vs compaction energy and to analyze these parameters at constant moisture contents, such as the optimum dry unit weight moisture content, or constant saturations. Figures 13 and 14 show dry density and unconfined compressive strength vs compaction energy. These plots generally indicate that at a constant saturation the parameters of dry density and unconfined compressive strength increase with an increase of compaction energy; however, the strength properties of a compacted soil as indicated by rheological parameters do not proportionately improve with the increase of compaction energy. This may be due to many factors, such as the nature of the

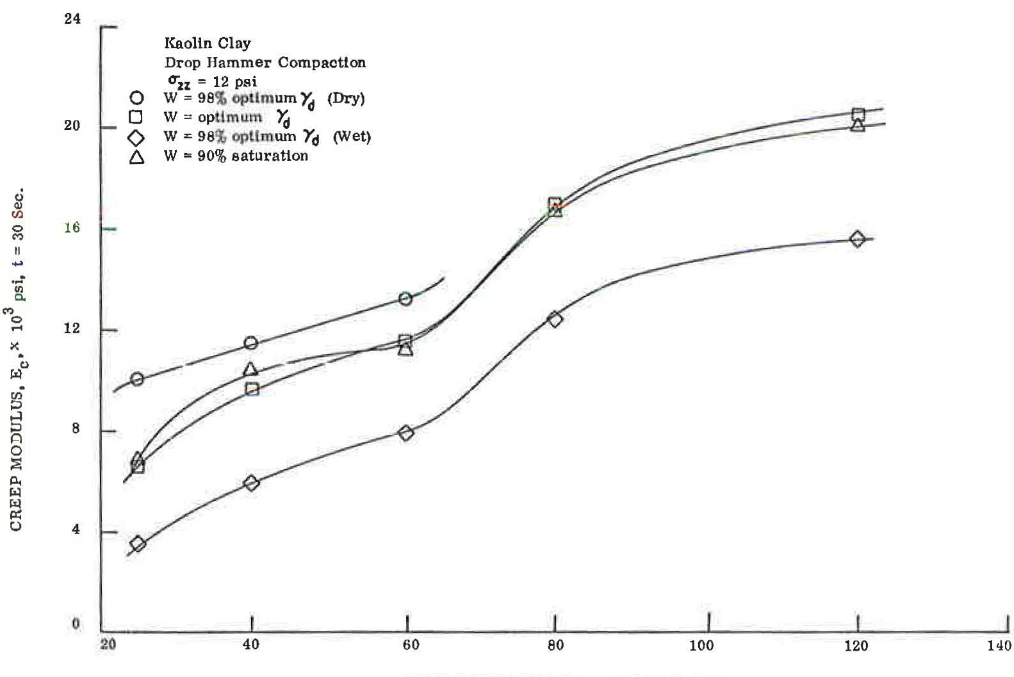


Figure 15. Creep modulus vs compaction energy.

structural arrangement of soil particles of the compacted soil and possible degradation of soil grains under high compaction energy, which were beyond the scope of this study of the phenomenological response of soils.

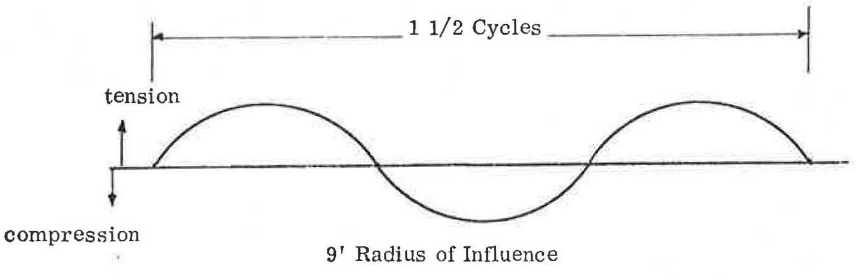
Figure 15 shows elastic creep modulus, E_c, vs compaction energy. The maximum possible value of E_c is the desirable characteristic. The optimum compaction energy found by the E_c criterion in the case of impact compaction is quite different from the optimum energy determined by the maximum dry density criterion, and would be the most economical compacted state to be desired in field construction for the particular soil, type of compaction and conditions investigated. If comparable experimental data can be obtained for the field strength-compaction energy relation of a soil it will be possible to expend less compaction energy on a given soil and obtain a more stable material for the construction of engineering structures.

The creep modulus parameter continues to increase with an increase in compaction energy. However, the rate of change in strength parameter is small beyond 80 blows compaction energy and, therefore, an optimum energy can be arrived at keeping in view the ratio between the increase of creep modulus and input of compaction energy.

From the creep data of axial strain as a function of time, using the electrical-mechanical analogy (9), it is possible to transform the creep moduli from the time domain to the frequency domain and calculate the magnitude of the elastic complex modulus, $|E^*|$, and its phase angle, ϕ_E, directly in the frequency domain. The parameters $|E^*|$ and ϕ_E can be evaluated for a particular frequency of loading to define the response of a viscoelastic material to any type of loading function. Table 2 indicates the approximate correspondence between the frequency of loading in the laboratory and the speed of a moving traffic load. Therefore, the complex parameters evaluated at any given frequency can be related to a corresponding traffic load moving at a particular speed (12). Figure 9 shows the parameters $|E^*|$ and ϕ_E at ω = 0.1 radians per sec vs compaction energy at constant moisture contents. Figure 16 shows $|E^*|$ vs

TABLE 2

TEST FREQUENCY AND VEHICLE SPEED CORRELATION



Vehicle Speed in mph	Time of Influence of One Cycle in Seconds	ω - Frequency in Radians/ Second
0.065	63	0.1
4	1.02	6.2
15	0.27	23.3
30	0.135	46.6

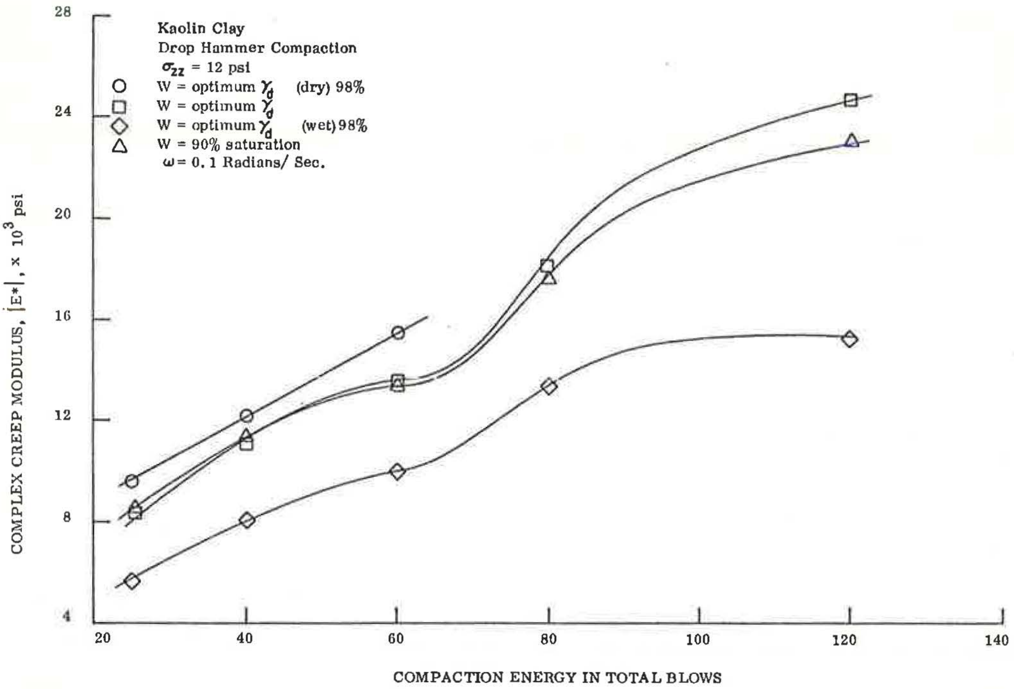


Figure 16. Complex creep modulus vs compaction energy.

compaction energy. The same optimum value of compaction energy can be evaluated using either E_c or $|E^*|$ as parameters, which in this case is 80 blows. The E_c compaction parameter is easily evaluated in laboratory tests and the optimum level of compaction energy is the same as the optimum obtained using $|E^*|$ and ϕ_E to evaluate soil compaction.

It was found that $|E^*|$ does not generally change significantly beyond a frequency of 0.1 radians per sec. However, the phase angle, ϕ_E, continues to vary slightly with frequency. Values of $|E^*|$ below a frequency of 0.1 radians per sec pass through a transition zone near this frequency. It should also be noted that this fundamental material property, E^*, is independent of the type of test and can also be directly utilized in pavement design procedures now in the research phase (12, 6). The parameters at a frequency of 0.1 radians per sec were plotted to evaluate the effectiveness of compaction, although any frequency could have been used. The desirable characteristics for the compacted soil, in the light of material science concepts, are a maximum value of $|E^*|$ and a minimum value of ϕ_E. The values of $|E^*|$ increase with an increase of compaction energy except for the higher moisture contents, but the rate of increase is not proportional to the increase in compaction energy and this should guide the engineer in selecting the optimum compaction energy.

In order to completely define the stress-strain response of a linear viscoelastic material, two material constants are required as in the case of the elastic theory. The second suitable material constant which can be evaluated experimentally is the complex transverse modulus, T^*, which relates axial stress and radial strain. This parameter was also utilized in this study on a limited basis. The complex transverse modulus can be evaluated by measuring the lateral strain. Thus, in the viscoelastic theory both $|E^*|$ and ϕ_E, $|T^*|$ and ϕ_T are the material constants necessary to define the state of a compacted soil.

In Figure 10 the response of a soil under load in a creep test was represented by an equation of the form $\epsilon = ct^m$. The values of c and m may be utilized as parameters to determine the optimum combination of moisture content and compaction energy.

SUMMARY

1. Compacted soils behave as linear viscoelastic materials within a given range of stress or strain depending on the environmental conditions and compaction methods. Soils compacted and utilized for subgrades under highway pavements are subjected to stresses and strains within the linear range noted by direct experimental testing. Therefore, as an engineering approximation, the response of compacted soils under highway pavements can be treated and studied as that of a linear viscoelastic material.

2. General experimentation is usually conducted on samples previously untested; however, the loading history influences the mechanical behavior of the soil. Compacted soils exhibit a form of mechanical conditioning and when experimental specimens are tested both the deformation and strength properties measured will vary with subsequent repetitions of loading and unloading, suggesting that a conditioning of the samples by the testing loads may yield more realistic results in the case of highway pavement studies.

3. The total deformation under load in the creep test could be separated into three components: (a) the instantaneous elastic, (b) the retarded elastic, and (c) the viscous. As an approximation, the elastic creep modulus evaluated at a loading time of 30 sec was used to represent the instantaneous or elastic response. Using the creep modulus at 30 sec loading time as a parameter, the compaction characteristics of soils were evaluated. In the case of drop hammer compaction the value of E_c increases with an increase of energy but the rate of increase is not proportional to the increase in the input of energy.

4. Analogous to Young's modulus, E, for an ideal elastic material the complex elastic modulus, E*, or the magnitude of the complex elastic modulus, E* , and the corresponding phase angle, ϕ_E, can be used as rheological strength parameters in evaluating the state of a compacted soil. The rheological parameters increase with increases of compaction energy, but not in the same proportion. From Figures 9 and 16 it can be concluded that 80 blows is the optimum compaction energy based on economic considerations.

5. In the stress relaxation tests, the desirable characteristic is a maximum value of $E_r(t)$. Eighty total blows per sample was the optimum compaction energy yielding a maximum possible value of $E_r(t)$ at a constant saturation.

6. The strain-time response in a creep test indicates that the axial strain and time are related by a power relation of the form $\epsilon = ct^m$ during the elastic and portions of the retarded elastic strain of the total response. During the steady-state flow portion of creep response the strain-time function can be described as $\epsilon = c' + m't$.

7. To rigorously define the stress-strain-time behavior of soils two material constants are required as in the elastic theory. The complex elastic modulus, E*, and a second modulus (which can be any one of the following: complex Poisson's ratio, V*, complex transverse modulus, T*, complex shear modulus, G*, and the complex bulk modulus, K*) will be sufficient to describe the true stress-strain-time response of any viscoelastic material. In many pavement design procedures, such as Westergaard's or Boussinesq's, an appropriate value of Poisson's ratio is often assumed and used in design techniques. Likewise an approximate and suitable value of V* can be assumed with the additional insight of laboratory tests of the measured lateral strains or volumetric changes. Using two material constants, the classical elastic equations can be used in the frequency domain (25) in predicting the performance of a subgrade under any type of loading. Once the optimum saturation or molding moisture content and the maximum possible or most economically desirable value of E*, T*, V*, etc., are determined, then a suitable type of compaction program in field can be evaluated to yield an optimum compacted soil.

The conventional soil compaction parameters, dry density and unconfined compressive strength, both increase with an increase of the compaction energy and are not fundamental properties of the material. It is established that there is an optimum level of compaction energy beyond which additional compaction will result in overcompaction of the material and a reduction in the overall strength of the soil. Therefore, using conventional soil strength parameters it may be difficult to find the most effective level of compaction energy. By verification of the linear viscoelastic response of soils under environmental conditions comparable to highway service conditions it is now possible to utilize rheological parameters such as E_c, $E_r(t)$, E^*, and others to evaluate soil compaction.

ACKNOWLEDGMENTS

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Field Study of Soil Compaction

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Full-scale field tests dealing with soil compaction for highway construction were undertaken to determine the following: (a) the desired characteristics of compacted soil, (b) how best to measure and specify the proper compaction, and (c) the effectiveness of various methods of achieving compaction. The test variables included were type of soil or base course material, moisture content, lift thickness, type of compactor, compactive effort, and number of roller coverages. Measurements of the soil properties were made using a cone penetrometer, 6-in. bearing plate, CBR apparatus, seismograph, portable nuclear moisture/density instrument, nuclear Road Logger and sand cone, together with conventional moisture content procedures. The experiments were divided into 6 sets, 3 for subgrade soils and 3 for base course materials, each incorporating some of the independent variables for different purposes. Statistical techniques were used for planning the experiments and analyzing the data. This paper describes the scope of the field tests, the plans and procedures and the type of information being obtained.

•IN 1964, the U. S. Bureau of Public Roads in conjunction with 14 states and Puerto Rico undertook sponsorship of a comprehensive study of soil compaction. The research comprised three parts. North Carolina State of the University of North Carolina at Raleigh was to evaluate the state of the art of compaction of soil and rock materials for highway purposes. The Ohio State University was to study fundamental properties of soils in the laboratory, on a rheological basis, to determine what the basic properties of soils in relation to soil compaction are. The third part of the compaction triology was undertaken by the IIT Research Institute.

The objectives of this study were to determine (a) the desired characteristics of compacted soil, (b) how best to specify and measure the proper compaction, and (c) the effectiveness of various methods of achieving compaction. The test plan consisted of full-scale field tests using commercial compaction equipment, laboratory investigations of soil behavior, and analysis and experiments as required to interpret the field results and develop the theory.

The purpose of this report is to describe the field tests so that the reader can better understand and assess the test results and conclusions from the study which are contained in two other papers published in this RECORD (1, 2) as well as those that may be published at a later date. Specifically, the paper describes the plans for the field tests, the factors considered in the design of the experiments, and the procedures followed in conducting the tests.

The field tests were conducted from June through October 1965 at Hazelcrest, Illinois. From the outset, considerable effort was devoted to a comprehensive review of the field test plans. Although the test objectives were reasonably well established, the best plan for obtaining them was not easy to determine because many factors, both

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technical and nontechnical, had to be considered simultaneously. Each piece of compaction equipment and each soil has its own unique characteristics. The test plan had to accommodate these differences and still provide a valid comparison between the results for each set of variables. Bias, such as produced by weather and unknown factors influencing the results, had to be averaged out. The thousands of possible combinations of independent variables involved in the study had to be reduced by some rational process to an amount which could be handled within the time and funds available. A tentative selection of the variables was made during contract negotiations to provide a starting point for the program. These were revised in light of further information gathered during the research. Pilot tests (3) to provide necessary information to facilitate proper planning of the main field test program were conducted at the Hazelcrest test site during the last two weeks of October 1964.

The general criteria for establishing the test plan were as follows:

1. Represent as broad a range of compaction equipment and soil conditions as possible.
2. Select lift thickness to cover the range of principal interest, 6 to 18 in.
3. Provide a variation in moisture content from dry to wet of optimum.
4. Choose methods of measuring soil properties to be indicative of the various important characteristics and at the same time be rapid and nondestructive, and have potential for use as construction control tests.
5. Consider methods of soil preparation.
6. Select the specific test variables to permit analysis of the results on a statistical basis, taking into account the large variability anticipated in the field.

The detailed test plans were prepared after an analysis of the pilot test results, discussions with the project steering committees and a review of the overall research objectives. These plans are presented in this paper together with a brief description of the apparatus and procedures used.

EXPERIMENTAL PLAN

The basic independent variables considered for inclusion in the field test plan were (a) subgrade soil and base course material types, (b) moisture content, (c) compaction equipment type, (d) level of compactive effort, (e) number of roller coverages, (f) lift thickness, (g) characteristics of foundation beneath lift being compacted, and (h) method of soil preparation.

It was intended that the range of variables selected be as broad as possible to permit a comprehensive study of the problem of soil compaction. The tests were conducted in the field under as realistic conditions as possible to permit the most direct and immediate application of the results to construction practice. Selection and grouping of the major variables were based on a valid statistical plan to provide the most information for the number of tests possible within the available funds, recognizing that large variability should be expected in the field. To satisfy these conditions, it was decided that replicate tests should not be included and that the size of the experiment, i.e., number of related tests, should be large enough to assure distinguishing real effects from random variations of the magnitude anticipated. The viewpoint was taken that a full factorial experiment would provide the best chance of success in interpreting the results.

A range of moisture content for each of the soils was considered essential for three principal reasons: (a) moisture content significantly influences soil properties, (b) optimum moisture content varies with compactive effort which is both different and unknown for each compactor, and (c) an understanding of field moisture-density relationships would be a significant aid in the accomplishment of the study objectives.

Compaction of subgrade soils most commonly involves multiple lifts. It is generally believed that during the compaction of the topmost lift the underlying lifts will also be affected and their properties in turn will influence the compaction of the topmost lift. However, even though these are important considerations, it was decided to eliminate them from the test plan in favor of other factors. This was done by constructing all

TABLE 1

CLASSIFICATION OF SUBGRADE SOILS

Name	Specific Gravity (G)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Color	Classification	
						Unified System	AASHO System
Clay	2.70	36.7	14.7	22.0	Tan	CL	A-6(13)
Silty clay	2.70	32.8	19.3	13.5	Gray	CL	A-6(9)
Silty sand and gravel	2.70	21.3[a]	14.0[a]	7.3[a]	Gray	SM-SC	A-4(1)
Silt	2.71	25.1	21.5	3.6	Tan	ML	A-4(8)
Sand	2.70	—	—	Nonplastic	Light brown	SP	A-3(0)

[a]Material passing No. 40 sieve.

test sections as single lifts on a prepared soil foundation whose strength was generally greater than that of the lift being compacted. The order of testing was randomized to help minimize any bias which might occur because of changes in the foundation during the summer. The major problem with the single-lift approach occurred with the sheepsfoot roller, because it usually will not compact the entire lift thickness. To provide some adjustment for this situation, the sheepsfoot lifts initially were made 2 in. thicker than those for the other compactors.

The selected test plan included the following test conditions:

1. Five subgrade soils (Table 1) and five base course materials (Table 2).
2. Four moisture contents for the subgrade soils, selected to bracket the estimated equipment optimum and the Proctor standard and modified optimums, and a single appropriate moisture content for the sand and the base course materials.
3. All test sections with as nearly as possible the same foundation conditions (the embankment lifts were constructed individually and then removed after final inspection).
4. Nominal loose lift thicknesses of 6, 12 and 18 in. for the subgrade soils and thicknesses of 6 and 12 in. for the base course series.
5. At least one piece of compaction equipment from each major category.
6. The use of a disk and a pulverizing mixer as alternatives for soil preparation.

Not all combinations of these conditions could be studied because of the extremely large number of tests this would require. Instead, six series of tests were designed, each incorporating some of the independent variables for different purposes.

TABLE 2

CLASSIFICATION OF BASE COURSE MATERIALS

Designation	Liquid Limit of Fines	Plastic Limit of Fines	Specific Gravity (G)	AASHO Gradation
Open-graded gravel or clean gravel	—	—	2.75	—
Dense-graded gravel with plastic fines or plastic gravel	—	18.1	2.76	A
Dense-graded gravel (PI = 0-6) or crushed gravel	14.8	Not Obtainable	2.76	B
Open-graded crushed stone or clean limestone	—	—	2.74	—
Dense-graded crushed stone or crushed limestone	17.0	Not Obtainable	2.77	B

The following basic types of compaction equipment were selected:

1. An intermediate pneumatic tire roller with variable wheel load and a heavy pneumatic tire roller.
2. An intermediate steel wheel vibratory roller providing two frequencies and a heavy vibratory roller.
3. A plate vibrator providing two frequencies.
4. A segmented pad roller with variable contact pressure.
5. One self-propelled sheepsfoot roller with variable foot pressure and one vibratory sheepsfoot.
6. One combination pneumatic-vibratory-smooth wheel roller for special tests.
7. A three-wheel smooth wheel roller with variable ballast.

The advice of a manufacturers' steering committee was sought in selecting the particular pieces of compaction equipment to satisfy these criteria. The specific operating conditions for each piece of equipment were established based on the instructions of the manufacturers. The general characteristics of the compactors are given in Table 3.

TABLE 3

COMPACTOR CHARACTERISTICS

Type	Designation	Conditions Used in Tests
Intermediate pneumatic	P1	Seven 11.00 × 20, 16-ply tires at 105 psi inflation pressure, 15,300 lb total weight, speed 3 mph, self-propelled.
	P2	Same except 24,000 lb total weight.
Heavy pneumatic	P3	Four 18.00 × 25, 24-ply tires at 70 psi inflation pressure, 37,800 lb total weight, speed 1.5 mph, towed by dozer.
	P4	Same except 52,700 lb total weight.
Intermediate vibratory	V	Smooth steel roller 75 in. wide by 47 in. diameter, static weight 10,500 lb, vibration frequency 1500 vpm, speed 1.5 mph, towed by dozer.
	V1	Same except 1400 vpm.
	V2	Same except 1600 vpm.
	V2S	Same except 1600 vpm, speed 1.0 mph.
	V4	Smooth steel roller 72 in. wide by 51 in. diameter, static weight 8100 lb, vibration frequency 2320 vpm, speed 1.5 mph, towed by dozer.
Heavy vibratory	V3	Smooth steel roller 78 in. wide by 60 in. diameter, static weight 21,700 lb, vibration frequency 1300 vpm, speed 1.5 mph, towed by dozer.
	V3M	Same except 1200 vpm.
Smooth wheel	SW1	Three steel wheels, rolling width 83 in., total weight 25,000 lb, speed 1.5 mph, self-propelled.
	SW2	Same except 31,400 lb total weight.
Combination	C1	Eight 7.50 × 15, 10-ply tires at 100 psi inflation pressure, 15,500 lb on tires, 26,900 lb total weight, speed 2 mph.
	C2	Same tires with 6900 lb weight followed by 72 in. wide by 32 in. diameter steel drum with 6900 lb weight, total weight 26,900 lb.
	C3	Same as C2 plus vibration at 100 vpm imposed on drum.
	C4	Drum only with 12,500 lb weight, 26,900 lb total weight.
Segmented pad	T1	Four steel wheels with segmented pads 44 or 69 sq in. each, 32,000 lb total weight, speed 3 mph, self-propelled.
	T2	Same except 34,600 lb total weight.
Plate vibrator	PV1	Six vibrating shoes at 400 lb static weight each, 4.4 sq ft area, frequency 2200 vpm, speed 1.5 mph, self-propelled
	PV2	Same except 4 shoes vibrating at 2900 vpm.
	PV2S	Same as PV2 except speed = 0.3 mph.
Vibratory sheepsfoot	SV	Drum 72 in. wide by 42 in. diameter with 112 feet 7 in. long at 7.5 sq in. contact area, static weight 9250 lb, frequency 2000 vpm, speed 1.5 mph, towed by dozer.
Sheepsfoot	S1	Twin-drums 5 ft wide by 5 ft diameter each with 120 feet at 7 sq in. contact area, static weight 21,000 lb on both drums, speed 3 mph, self-propelled.
	S2	Same except 30,000 lb weight.

The following types of measurement on the compacted materials were included:

1. Penetration resistance vs depth with cone penetrometer.
2. Deflection under load applied to 6-in. circular bearing plate.
3. California Bearing Ratio.
4. Moisture and density with portable backscatter nuclear instruments.
5. Moisture and density logs with a nuclear Road Logger.
6. Density with sand cone and moisture content using oven-dried samples.
7. Seismic velocity with seismograph.
8. Lift thickness before and after compaction.

In addition, measurements were made periodically on the foundation soils. Modified Proctor and CBR tests were made in the field on samples from each subgrade soil lift, and ambient temperature and humidity were recorded.

SUBGRADE SOILS

The combination of variables investigated in the subgrade soil tests is given in Table 4. These were accomplished in 3 test series for convenience. Since series 1 initiated the field test program, it served in part as a check on procedures before embarking on series 2, the main series of subgrade soil tests. Series 3, dealing only with sand, was conducted last. These tests provided 336 test sections to represent the soil types, lift thicknesses and compaction methods of Table 4, together with variation in moisture content, compactive effort and method of soil preparation.

For analysis the test sections were grouped in several ways. Series 2, the principal subgrade soil series, provided 256 test sections combining the following variables:

1. Four soils—moderately plastic clay, silty clay, silty sand and gravel, and silt.
2. Lift thicknesses of 6 and 12 in.
3. Four moisture contents.
4. Four compactors—intermediate pneumatic, intermediate vibratory, segmented pad and self-propelled sheepsfoot.
5. Two levels of compactive effort for each roller.
6. Soil preparation by pulverizing mixer.

This series resulted in a basic body of statistically meaningful data on which most of the analysis and evaluation of soil properties and measurement techniques for subgrade soils was based.

A second subset provided 36 test sections from series 1 and 2 combining (a) silty clay; (b) four moisture contents; (c) lift thickness of 12 in.; (d) nine different pieces of compaction equipment, each at a single level of effort (all equipment in Table 3 except plate vibrator); and (e) soil preparation by pulverizing mixer. This series permitted observing, for a single soil type and lift thickness, whether or not there would be any fundamental differences in conclusions for a wide range in type of compaction equipment.

A third subset of 24 test sections from series 1 provided information on methods of soil preparation by comparing the results using disking vs pulverizing mixer preparation for (a) heavy pneumatic roller on silty clay; (b) heavy vibratory roller on silty sand and gravel; (c) combination roller on silty clay; (d) lift thickness of 12 in.; and (e) four moisture contents.

A fourth subset of 16 test sections from series 1 provided information on thick lifts by combining (a) lift thickness of 18 in.; (b) two rollers—heavy pneumatic and heavy vibratory; (c) two soils—silty clay, and silty sand and gravel; and (d) four moisture contents. Because the pulverizing mixer could not handle 18-in. lifts, preparation was accomplished by adding and disking three 6-in. thick layers of soil.

A fifth subset of 16 test sections from series 3 provided information on compaction of sand by combining lift thicknesses of 12 in. and 18 in. and eight pieces of compaction equipment (all in Table 3 except the two sheepsfoot rollers).

TABLE 4

SUBGRADE SOIL TESTS

Soil Type	Loose Lift Thickness (in.)	Compaction Method: Pneumatic Inter.	Pneumatic Heavy	Vibratory Inter.	Vibratory Heavy	Combination	Segmented Pad	Plate Vibrator	Smooth Wheel	Sheepsfoot Vibr.	Sheepsfoot Self-Prop.
Moderately plastic clay	6	○ 8		○ 8			○ 8				○ 8
	12	○ 8		○ 8			○ 8			□ 4	○ 8
	18										
Silty clay	6	○ 8		○ 8			○ 8				○ 8
	12	○ 8	□ 8	○ 8	□ 4	□ 12	○ 8		□ 4	□ 4	○ 8
	18		□ 4		□ 4						
Silty sand and gravel	6	○ 8		○ 8			○ 8				○ 8
	12	○ 8		○ 8	□ 8		○ 8				○ 8
	18		□ 4		□ 4						
Silt	6	○ 8		○ 8			○ 8				○ 8
	12	○ 8		○ 8			○ 8			□ 4	○ 8
	18										
Sand	6										
	12	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1		
	18	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1	△ 1		

Note: Numbers indicate amount of test sections for each combination required by planned variation in moisture content, compactive effort and method of soil preparation.

□ Series 1 ○ Series 2 △ Series 3

Finally, some additional special tests from series 1 are included for general information. These were vibratory sheepsfoot on 12-in. lifts of three soils (clay, silty clay and silt) at four moisture contents.

BASE COURSE MATERIALS

The combinations of variables investigated in the tests on base course materials are given in Table 5. Ninety test sections are represented. Moisture content was not a control variable as in the subgrade soil tests. Instead, one replicate of each test section was provided at the moisture content existing in the materials as they were brought from the quarry stockpiles.

The tests were planned in 3 series (Table 5), but conducted simultaneously for convenience. The results are grouped into 3 subsets for purposes of analysis. The principal one provides 64 single-moisture test sections from series 5 combining the following variables:

1. Two base course materials—dense-graded gravel and dense-graded crushed stone.
2. Lift thicknesses of 6 and 12 in.
3. Four compactors—intermediate pneumatic, intermediate vibratory, plate vibrator and smooth wheel.
4. Two levels of effort with each compactor.
5. One replicate of each set of conditions.

This series provides the basic body of data on which the analysis and evaluation of soil properties and test techniques for the base course series will be based.

A second subset of 16 test sections from series 5 and 6 provides a comparison across all equipment types for one soil type by combining (a) one material—dense-graded crushed stone; (b) lift thickness of 6 in.; (c) all eight compactors in Table 5; (d) one level of effort with each compactor; and (e) one replicate of each set of conditions.

The third subset consists of 30 test sections from series 4 and 5 to compare the results with all five base course materials by combining (a) all five materials in Table 5; (b) lift thickness of 6 in.; (c) three compactors—intermediate pneumatic, intermediate vibratory and plate vibrator; (d) one level of effort with each compactor; and (e) one replicate of each set of conditions.

TABLE 5

BASE COURSE TESTS

Material	Loose Lift Thickness (in.)	Compaction Method							
		Pneumatic		Vibratory		Combination	Segmented Pad	Plate Vibrator	Smooth Wheel
		Inter.	Heavy	Inter.	Heavy				
Open-graded gravel or clean gravel	6	□ 2		□ 2				□ 2	
	12								
Dense-graded gravel with plastic fines or plastic gravel	6	□ 2		□ 2				□ 2	
	12								
Dense-graded gravel (PI = 0-6) or crushed gravel	6	○ 4		○ 4				○ 4	○ 4
	12	○ 4		○ 4				○ 4	○ 4
Open-graded crushed stone or clean limestone	6	□ 2		□ 2				□ 2	
	12								
Dense-graded crushed stone or crushed limestone	6	○ 4	△ 2	○ 4	△ 2	△ 2	△ 2	○ 4	○ 4
	12	○ 4		○ 4				○ 4	○ 4

Note: Numbers indicate amount of test sections for each combination required by planned variation in compactive effort plus one replication of conditions.

□ Series 4 ○ Series 5 △ Series 6

SEQUENCE OF OPERATIONS

A complete review of the sequence of operations employed in all 6 test series is beyond the scope of this paper. However, to provide insight into the order and coordination of tests, the procedures used during test series 2 (the main series) are outlined. In this series each individual lift was 160 ft long by 15 ft wide and divided into 4 equal test sections oriented end-to-end, each with a different moisture content (Fig. 1). Location flags were used to mark the position at which measurements were to be taken.

The pulverizing mixer was used for all soil processing to provide uniform distribution of moisture and to avoid undesirable density gradients in the loose lift material. Immediately prior to initiating compaction, moisture samples were taken from one location in each test section, with one 300-gram sample taken for each 3 in. of lift thickness. Simultaneously, two 1-gal cans of soil were taken from the same location in each section for modified Proctor compaction tests and unsoaked CBR tests. The tests were carried out during the time the lift was being compacted and tested. In addition, approximately 6 gal of soil were taken randomly along the lift and stored in 50 gal drums to acquire during the test program a large sample of material representative of each stockpile.

Two coverages, usually consisting of single passes, were then made with the selected compactor. At the completion of the second coverage, measurements were made with the portable nuclear, Road Logger, seismic, penetrometer and plate apparatus. With the exception of the Road Logger, all measurements were made once in each test section at the same predetermined random location. The Road Logger made one pass along the length of the lift within the compacted width. Two more coverages were then made with the compactor and the testing process repeated at a new random location. The remaining sets of measurements were made after 8 and 16 coverages.

On completion of the growth measurements after the 16th coverage, lift thickness measurements were taken. Then the lift was stripped down approximately 2 in. for a

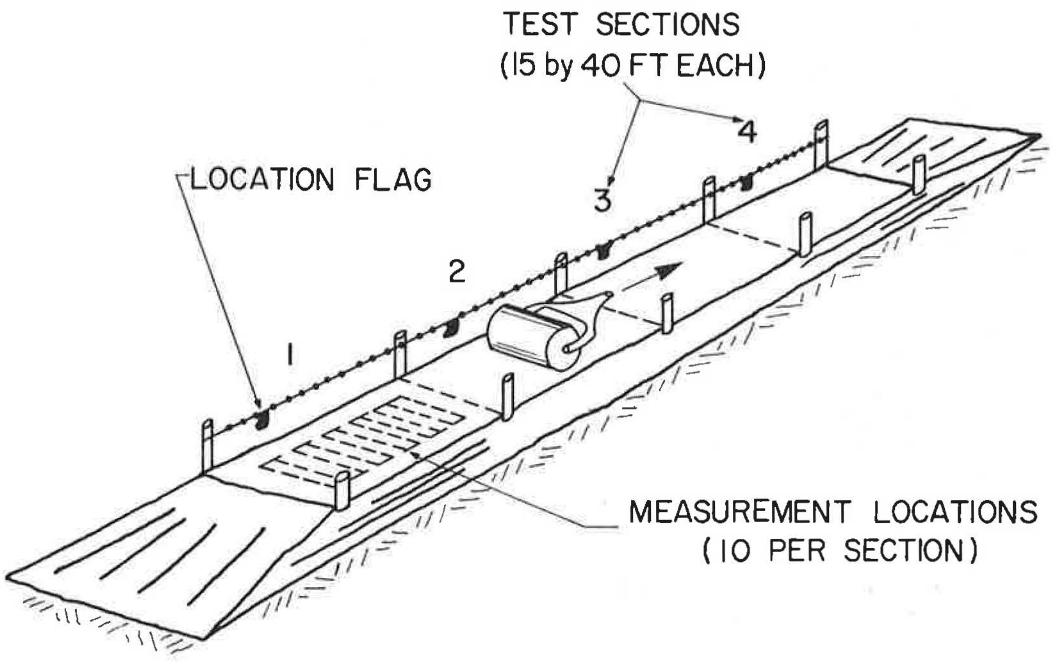

Figure 1. Lift layout with four test sections.

6-in. lift or 5 in. for a 12-in. lift and additional thickness measurements made. In general, the stripping operation was completed in about 20 min.

Final inspection tests were then performed. These included all previous measurements, and in addition, CBR tests in each test section, sand cone tests in sections 1 and 2 and moisture content determinations in each test section. The sand cone tests and moisture samples were taken at the exact location at which the final hand nuclear measurements were made. After these tests were completed, the remainder of the lift was removed and the test area prepared for another lift.

Approximately 25 to 30 min were required for each set of growth measurements, and about 45 min for final inspection measurements. Compaction and stripping time consumed approximately 50 min. Total time required for a test totaled approximately 3½ hr. Instrument and equipment malfunctions and inclement weather frequently increased the required time.

PREPARATION PHASES

Test Area

Preparation of the test site was started in September 1964 prior to the pilot studies. The area was cleared and graded and the 5-acre portion to be used for the test sections was inspected for suitability as a foundation for the compacted lifts. Pockets of unsatisfactory soil were removed and replaced with a moderately plastic clay so that the entire 5 acres would be relatively uniform in composition.

The clay, silty clay, and silty sand and gravel used in the tests (Table 1) were obtained from selected borrow pits at the test site. About 3000 cu yd of each of these materials were stockpiled on the site and then mixed by a bulldozer to insure homogeneity. The silt was hauled to the site from a location about 10 miles away. It was also mixed by a bulldozer. The sand was obtained from a location near Lake Michigan. It was very uniform and required no further processing.

The base course materials were hauled to the site from quarries in Thornton and Joliet, Illinois, and were deposited directly in the test areas as required for compaction. It was felt that the quarry control would be sufficient to assure adequate uniformity between test sections.

At the completion of site preparation, eight 30 by 120-ft test areas were laid out side by side. Soil within these areas was compacted by 18 coverages with a self-propelled sheepsfoot roller followed by 2 coverages with a 50-ton pneumatic roller. Each of the 8 test areas was then divided into four 30 by 30-ft test sections. This arrangement was made to provide up to four different moisture contents in each lift with the same soil. By putting all four levels of moisture in a single lift for the subgrade soil tests, moisture increments could be more precisely controlled. After subgrade soil series 1, the test sections were extended in length to 40 ft and reduced in width to 20 ft. This provided a more suitable layout for preparing and compacting the soil.

Measurements of penetration resistance, bearing plate, and CBR were then made in each test area to obtain quantitative measures of the foundation conditions. These measurements were repeated periodically to determine variations in conditions. After a general increase in penetration resistance in all sections for the first month, fairly uniform results were obtained thereafter.

Subgrade Soil Processing

It was desired that the stockpiled soils be uniformly dried to a moisture content equal to or less than the lowest of the four preselected test values for each lift before placing them in a test area. Drying was best accomplished at the stockpile, but some drying had to be accomplished after the soil was placed in the test area. In addition, a compromise had to be found between the amount of time which could be devoted to drying the soil and the minimum moisture content acceptable. In general, the minimum moisture content tested was 2 to 3 percent higher than the preselected desired

TABLE 6

RELATIONSHIP BETWEEN SOIL TYPE AND MOISTURE CONTENT

Soil Type	Range from T-99 to T-180 Optimum Moisture (%)	Nominal Moisture Content (%)				Moisture Increment (%)
		1	2	3	4	
Clay	10-16	8	11	14	17	3
Silty clay	9-13	7	9	11	13	2
Silty sand and gravel	7-10	5	7	9	11	2
Silt	11-13	7	9	11	13	2

minimum. This was considered acceptable, since the optimum moisture content for each compactive effort would still not be generally exceeded.

Prior to placing the soil in the test areas, elevation marks were placed on the grade stakes defining the corners of each test section. Reference elevation marks were obtained by means of a stringline stretched across the section between stakes and adjusted to an average height of 6 in. greater than the planned lift thickness. This was a rapid and accurate method of setting elevation guide marks.

Soil was transported from the stockpile to the test areas with a self-loading scraper. For test series 1 the scraper laid the soil in two parallel strips. The motor grader then spread the soil to the proper width and leveled the surface. Additional soil was placed, spread, and leveled until the desired loose thickness was obtained over a width sufficient to accommodate at least two adjacent nonoverlapping passes with the particular compactor being tested, allowing a minimum of 2 ft of shoulder on each side. The scraper and grader operators were instructed to follow the same wheel paths when traversing the lift each time to avoid compacting the material in the areas where the test measurements were to be made. This was particularly important in spreading the second and third layers of the 12- and 18-in. lifts when prepared with the disk.

For series 2 the soil was deposited in a single lane by the scraper and spread to a width of 12 to 15 ft by the motor grader. After a sufficient amount of soil had been deposited, stringline measurements were made and the final grading carried out to provide the desired lift thickness. Enough additional soil was placed at the ends of each lift to provide 10- to 15-ft long ramps.

After arriving at estimates of initial moisture content and wet density, the number of cubic feet of water to be added to each test section was calculated. The selected nominal moisture contents for each soil were as shown in Table 6. The moisture order assigned each section within a lift varied randomly as prescribed by the statistical plan instead of increasing monotonically from one end of the lift to the other.

Three techniques were used for mixing the water and soil: (a) surface sprinkling followed by a towed disk, (b) surface sprinkling followed by a pulverizing mixer, and (c) direct addition of water during processing by a pulverizing mixer. Sprinkling and disking was found to be unsatisfactory for mixing water into the more cohesive soils due to the immobilization of the disks because of clogging by soil and the tendency, in the wetter sections, for soil to be transported from one test section to another. Sprinkling and pulverization proved to be the most advantageous so it was used for most of the subgrade soils. Direct addition of water during pulverization, though most desirable, could not be satisfactorily accomplished because of a variety of difficulties with the water dispensing system on the mixer.

SOIL TESTING INSTRUMENTS

The instruments used for measuring soil properties are briefly described here; additional details are given elsewhere (1, 3, 4, 5).

Figure 2. Penetrometer apparatus.

Figure 3. Bearing plate apparatus.

Penetrometer

The field penetration apparatus was designed to measure and record continuously the penetration resistance vs depth (Fig. 2). The penetrometer used was the standard Corps of Engineers 30 deg (included angle) cone with a ½-sq in. base area. The rate of penetration was maintained at a constant value of 2 in./sec by the hydraulic control system. The maximum depth of penetration was 15 in. No preparation of the soil was required and the total measurement time was less than 1 min.

Bearing Plate

A rapidly loaded 6-in. diameter plate (Fig. 3) was developed in which load could be automatically applied and removed at a controlled rate of 500 lb/sec, and a continuous record of load vs deformation obtained. A maximum load of 4500 lb was provided using the weight of the test vehicle as a reaction. If a sinkage of 1 in. was developed the load cycle was automatically terminated. Prior to the test, uncompacted soil had to be removed and the surface smoothed. Displacement was referenced to the soil surface by a special mechanism.

Field CBR Test

The CBR apparatus (Fig. 4) included a piston, surcharge weights, and a loading device. The piston was 5 in. long and had an end area of 3 sq in. (diameter of 1.95 in.). A 6-in. diameter plate was fixed to the top end of the penetration piston to permit it to be clamped to the bearing plate apparatus.

Three annular steel rings were used to produce the same surcharge pressure as the 20-lb weight for the laboratory CBR test specimens. The loading device consisted of a hand pump connected into the hydraulic system. This pump allowed load to be applied at slower rates than provided by the automatic control system of the bearing plate.

Figure 4. Field CBR apparatus.

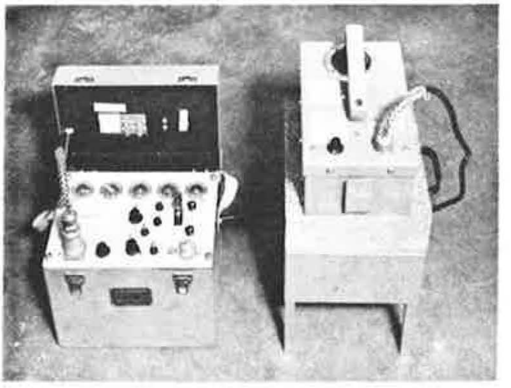

Figure 5. Portable nuclear moisture/density apparatus.

Portable Nuclear Moisture/Density Instruments

Two model 5901 d/M combination backscatter moisture-density gages (Fig. 5) were furnished for use on the compaction field studies by the Nuclear Chicago Corp. This gage has a single 4-mc radium beryllium source which emits both gamma rays and neutron radiation permitting measurement of both moisture and density. A two-position switch on the gage housing provides for selection of either moisture or density measurement.

A series of tests was conducted in the laboratory prior to entering the field for the purpose of becoming familiar with the characteristics of the instrument, and for establishing calibration curves for the soils used in the study. The calibration curves were checked against those provided by the manufacturer, and where significant differences existed the calibration curves developed in the laboratory took precedence.

Standard blocks of material were used to provide reference measurements of wet density and moisture density (moisture content in lb/cu ft), and to provide a continuous check on the operation of the gage and scaler. The gage seating technique used a cushion of dry sand following the removal of loose soil and trimming of the surface. Three 1-min readings were then obtained on the lift for both moisture and wet density and divided by the average of respective sets of reference counts taken before and after the measurements on the soil. The moisture density and wet density were determined from the count ratio by using the appropriate calibration curve.

Nuclear Road Logger

The Road Logger (Fig. 6) is a specially designed vehicle manufactured by Lane-Wells Co. that is equipped to record the wet density and moisture density of the material over which it is driven. Either stationary or continuous moving logs at driving speeds up to 3 mph may be obtained. Measurements are recorded on a strip chart presenting direct reading of wet density and moisture density vs distance of travel.

The moisture density and wet density measuring systems are mounted on two-wheel carriages which rest on the ground surface when logging in order to minimize the variations of surface roughness. The system investigates a single track along the ground surface approximately 12 in. wide. Material to a depth of approximately 8 in. influences the reading, but the measurement is highly weighted toward the surface since the influence of soil diminishes exponentially with depth. When logging, the moisture

Figure 6. Road Logger apparatus.

Figure 7. Seismic apparatus.

density and wet density plotted is a continuous average of the values over a preset distance of past travel, generally 6 ft. No soil preparation was needed except for the lifts compacted with the sheepsfoot rollers.

Seismograph

A seismograph was used to determine rapidly the velocity of impulses through the compacted layers. Theoretically, seismic wave velocity is related to the density and elastic properties of the soil.

The instrument selected was a Minnetech Labs Model MD-3 (Fig. 7) modified to permit measurements over horizontal distances of 3 to 24 in. It consisted of four essential parts: (a) counter, (b) triggering device, (c) sensing device, and (d) a means to induce an impulse into the soil. A metal bar or spike held firmly on the ground surface was struck with a hammer. The triggering circuit was arranged so that the circuit was closed (starting the counter) when the hammer made contact with the bar or spike. A geophone, anchored to the soil by either a flat plate or spike, served as the sensing device.

CONCLUSIONS

The purpose, scope and operational procedures used in the compaction study have been described to provide the necessary background for understanding and appreciating the results of this study. More detailed information on the test plan, compaction and testing equipment, and test procedures are given elsewhere (3, 4, 5).

Over 10,000 measurements were taken and are now in the process of analysis. Some of the results are presented in two companion papers in this RECORD (1, 2). Even with the aid of statistical methods and computer techniques, the complete realization of the potential of the data will not be realized for some time, since a considerable amount of analysis is required for proper interpretation. In addition, it should be recognized that in spite of the great mass of data collected thus far, more research is required both in the field and in the laboratory to supplement data previously collected. This is necessary to broaden the scope of understanding of all aspects of the compaction process and the properties of compacted soils.

ACKNOWLEDGMENTS

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Evaluation of Rapid Field Methods for Measuring Compacted Soil Properties

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One of the objectives of a recently completed field test program was to study rapid methods of measuring properties of compacted soils for purposes of construction control. A number of techniques for determining density, strength and stiffness characteristics of soils were evaluated in tests providing a range of soil types and a variety of compaction methods. Commercially available devices were (a) a portable backscatter nuclear moisture/density instrument, (b) a nuclear Road Logger for measuring moisture and density, and (c) a seismograph. In addition, apparatus was designed and constructed for rapidly measuring penetration resistance using a cone penetrometer, stiffness using a 6-in.-diameter bearing plate, and CBR. For comparison, conventional sand cone and moisture content measurements were obtained. The results show the suitability of the devices for detecting changes in compaction with compaction effort, the correlation between the different methods and the variability of the observed properties.

•ONE objective of a full-scale field compaction study recently undertaken was to evaluate rapid nondestructive tests for measuring properties of compacted soils. Compaction specifications are usually based on an optimum moisture content and maximum dry density as determined from standard laboratory compaction tests. Compaction control is based almost solely on field measurement of density, which generally is obtained by the sand cone or rubber balloon test, with nuclear measuring techniques coming into greater prominence each year.

With the exception of nuclear techniques, which are not yet fully accepted, field and laboratory tests are generally too time-consuming to keep pace with present construction methods. In addition, it is quite possible that compaction specifications should actually be based on some property other than density; i.e., some other property (or properties) might provide a more direct measure of the important performance capabilities of the compacted soils.

Hampton and Selig (1) discuss the scope of the compaction study, the field test plan, and the equipment and apparatus used. Selig and Truesdale (2) discuss the influence of the test variables on compaction and the measured properties of the compacted material. This paper describes the several field methods selected to measure the moisture, density, strength, stiffness and seismic wave velocity of soils, and discusses their advantages and limitations as rapid, nondestructive tests and the degree to which the measured properties correlate. Measurements made were: (a) moisture and density with a portable nuclear gage, (b) moisture and density with a mobile nuclear logging device, (c) moisture and density with sand cone apparatus, (d) penetration resistance, (e) plate bearing stiffness, (f) seismic wave velocity, and (g) the California Bearing Ratio. In addition, samples were taken from each test section for determination of moisture content by oven-drying methods.

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

DESCRIPTION OF APPARATUS

Hampton and Selig (1) briefly described each type of apparatus used in the field tests and presented illustrations of their operation in measurement applications. This paper deals primarily with the techniques employed in using the apparatus, the problems encountered and the time required for measurements.

Portable Nuclear Instrument

The portable nuclear instruments used were two Nuclear-Chicago Model 5901 d/M gages with Model 2800 A scalers. This gage is a backscatter instrument permitting measurement of either wet density or moisture density. (Moisture density is defined as moisture content in lb/cu ft of water.) Standard blocks of material were furnished to provide reference measurements for density and moisture in order to check the overall operation of the probe and scaler.

The gages were calibrated in the laboratory on soil samples taken from the stockpiles used for the field tests. Calibration curves were established in terms of count ratio, and all measurements made in the field were converted to count ratio. The count ratio was obtained by dividing the backscatter measurement on the soil by the counts obtained for the instrument on the appropriate moisture or density reference block. During the field tests, sets of five 1-min counts on these blocks were obtained at 2- to 4-hour intervals for both moisture and density. The average of three 1-min counts was obtained on the soil. The count ratio was calculated by dividing this measurement by the average of the two sets of standard counts taken before and after the field measurement.

The purpose of the count ratio is to compensate for variations in counts caused by such effects as temperature, location, and/or time on the instrument. The assumption is that the percentage change in counts produced by these undesirable effects is the same for the soil measurements and the reference readings, so that the count ratio will be unaffected. The degree to which this was true in the field tests can be determined by analysis of the test data with and without dividing the soil measurements by the reference counts. This has yet to be done, but there is evidence that unless the instrument effects are pronounced, the use of count ratio does not improve accuracy.

In performing the field measurements, soil was removed to the depth of disturbance caused by the compactor, e.g., below the depths of foot penetration with a sheepsfoot or below rut depth with a pneumatic compactor. The soil surface was leveled and a thin layer of sand spread on the surface. The gage was then placed on the sand bed and rubbed down into firm contact with the soil.

The most general criticism of portable backscatter instruments is the limited depth of soil involved in the measurement. No direct study was made of this problem on the compaction study, but it is generally acknowledged that the measurement is highly oriented toward the surface with the majority of the backscatter counts being determined by the top 1-in. layer of soil. The gage is also sensitive to seating techniques and soil surface conditions.

Aside from soil surface preparation, the gage used was simple and easy to operate and had very little downtime throughout the entire test series. A set of measurements, i.e., moisture and density, at a given location required approximately 12 min. This involved three 1-min readings each for moisture and density, totaling 6 min, with the remainder of the time required for surface preparation and data recording.

During the study a variety of portable nuclear gages representing most of the manufacturers were given preliminary evaluation. Three major problem areas were encountered. First, in a high percentage of cases the instruments required adjustment before reliable operation was obtained. Second, in almost no instances were the calibration curves provided with the instruments in agreement with those obtained on this study, and the measurements at the same locations with the different instruments were not the same. Third, suitable operating procedures were not available and the opinions of manufacturers and users with respect to measurement accuracy and proper techniques varied widely.

Road Logger

The Road Logger is a development of the Lane-Wells Co. which detects backscattered radiation to provide measurement of soil moisture and density. The measurements are recorded in the form of a strip-chart plot (Fig. 1) which gives a direct reading of average wet density and moisture density vs distance of travel. The moving logs represent continuous average measurements through integration of the count rate over a fixed distance of past travel. A single operator drives the vehicle and monitors the recording instruments.

The nuclear probes are mounted on two-wheeled carriages. During measurements, the carriages are lowered until their wheels touch the ground, thus providing a controlled gap between the probe and soil surface. Material to a depth of 8 in. is reported to influence the measurements, but, as with the portable backscatter gages, the measurement is weighted toward the surface because the percentage of backscattered radiation detected diminishes exponentially with depth.

Although the general principles of this instrument are similar to those of the portable nuclear instrument, the Road Logger embodies several features which reduce the effects of the undesirable factors influencing the behavior of nuclear instruments. By appropriate shielding, the emitted and detected radiation is collimated so that the Road Logger reading is affected by a greater depth of soil and is less weighted toward the surface. The controlled gap, or standoff, reduces the effect of surface roughness and eliminates the seating error present when no gap is used. For example, the creation of a $\frac{1}{16}$-in. gap for the portable instrument will cause about a 9-pcf change in density, whereas a $\frac{1}{2}$-in. change in gap for the Road Logger will cause only about a 1-pcf change in density. Standoff variation with the Road Logger will occur if the carriage wheels ride over an uneven soil surface; however, as long as the average gap remains constant over the integration distance this will produce minimal error.

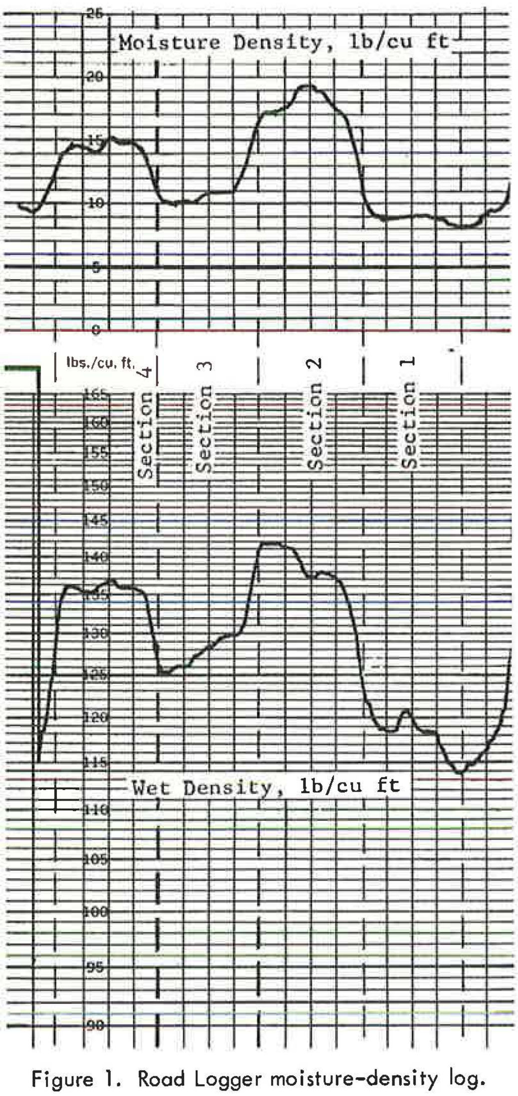

Figure 1. Road Logger moisture-density log.

Finer discrimination of energy level of detected radiation with the Road Logger is believed to reduce the effect of soil composition on the moisture and density readings. Instrument variations with time and ambient conditions can be minimized with the Road Logger by periodic checking with the built-in calibration blocks. A calibration check can be accomplished in a period of 1 to 5 min depending on the extent of adjustment required.

In general, the Road Logger was found easy to operate. It does, however, require assignment of relatively high caliber personnel, and with the particular units used on the field test program considerable downtime was encountered due to mechanical and instrumentation difficulties. The Road Logger is not suitable for use with compactors such as the sheepsfoot which do not provide a firm surface. However, with proper procedures uneven

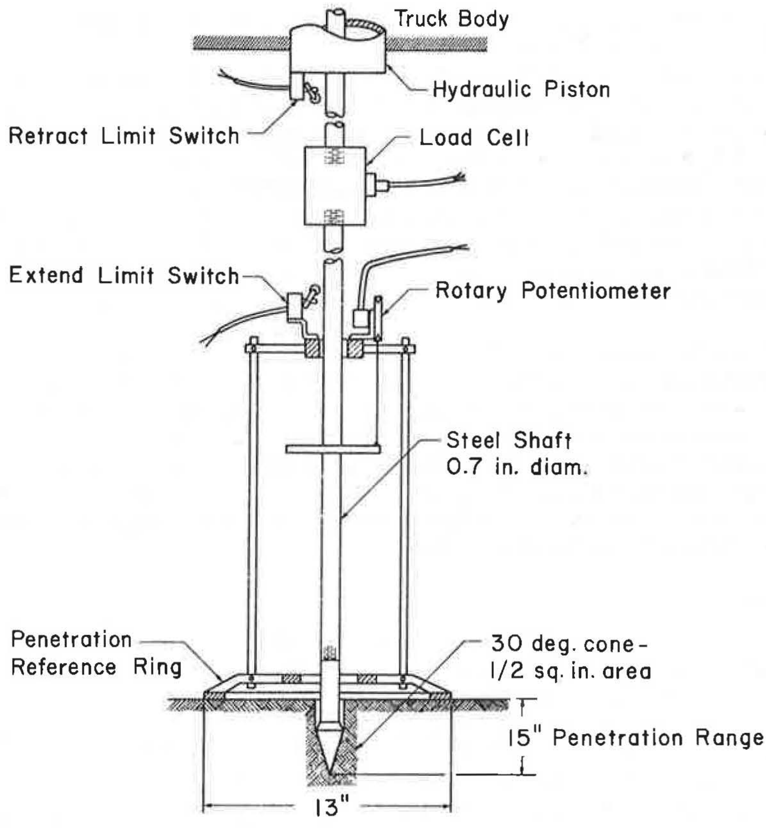

Figure 2. Penetrometer mechanism.

surfaces can be handled. Perhaps the biggest advantage of the Road Logger is that the uniformity of density and moisture over large areas can be quickly determined.

Penetrometer

The penetrometer apparatus used in the field tests was developed at IITRI (Fig. 2). It incorporated the standard Corps of Engineers 30-deg cone tip with a ½-sq in. base area. The electrohydraulic supporting system was designed to provide a constant rate of penetration of 2 ips independent of the resistance of the soil. The force required to cause this rate of penetration was plotted on an x-y recorder providing a continuous record of resistance vs distance of penetration throughout the compacted layer. The apparatus was mounted on a truck to provide a portable reaction frame.

The steel penetrometer shaft slides vertically through the center of a ring mechanism which rests on the ground surface to serve as the penetration displacement reference. The joints of the mechanism were designed to permit the ring to adjust for an uneven soil surface without affecting the displacement measurement or the movement of the penetrometer. The shaft was relieved above the cone in an attempt to eliminate friction between it and the hole made by the cone.

When retracted for traveling, the ring and cone were suspended about 15 in. above the ground surface. At the beginning of a measurement the entire assembly was lowered until the ring touched the ground and seated itself. The penetrometer shaft continued to move downward at a constant rate, unwinding a cable connected to a potentiometer mounted on the ring assembly. In this manner no penetration was recorded until the ring was positioned on the ground. When the cone penetrated the soil to a depth of 15 in., a limit switch was activated causing the penetrometer to retract. After retracting to its initial above ground position, a second limit switch ended the cycle. The total cycle time was about 1 min.

One of the primary advantages of this device was its independence of operator technique. The system was completely automated and required only minimal surface preparation. The device was the most reliable in operation of all the instrumentation used, having only a slight amount of downtime throughout the test series.

The primary problem encountered with the measurement was data interpretation. Peak value of penetration resistance is easy to discern from the records, but this was not felt to be the most representative measure of lift penetration resistance. The average penetration was believed to be most representative, but judgment was required to establish the average value because of variation in resistance over the depth of the layer.

The penetrometer was rugged enough to withstand the interference of rocks and gravel in the soil, but the readings are not considered meaningful in base course materials or soils with high gravel content. This was the only device used in the tests which could examine the uniformity of the compacted layer throughout its thickness and provide a representative average. It was also the only method independent of surface conditions. An examination of the data suggests that side friction on the penetrometer shaft was probably not negligible, hence subsequent apparatus should provide a force sensor at the tip of the penetrometer.

Bearing Plate

The plate bearing test provided a measure of soil bearing strength and stiffness. In the standard field bearing test, a 30-in. diameter plate is slowly loaded while measurement is made of plate sinkage. Such a test is obviously not suitable for compaction control. For the compaction field tests, a rapidly loaded 6-in. diameter plate was used instead. One advantage was that plate response was primarily a measure of individual lift strength rather than some composite measure of lift and foundation strength as would be obtained with a large-diameter plate. Theoretical computations and field experience indicate that for short-term loading, a condition present here, the region of significant stress increase extends down to a depth roughly two times the width of the loaded area. Thus the effective depth of measurement for the 6-in. plate is approximately 12 in., but weighted toward the surface, while the effective depth of the 30-in. plate is 60 in. The other advantages were that the reaction load requirements were compatible with vehicular mounting for portability and that the test could be performed rapidly.

A schematic of the bearing plate apparatus developed by IITRI for use in the field tests is shown in Figure 3. A 6-in. diameter steel plate is attached to the end of a steel shaft by a swivel joint to permit it to adjust to the slope of the ground surface. A crossarm is attached to the loading shaft by a bushing and swivel joint so that the reference feet, which also swivel, can adjust to the soil surface. An electrohydraulic system was used to provide a prescribed time-dependent load on the plate. The load-sinkage output was displayed on an x-y recorder.

Prior to the seating of the plate, the ground surface was leveled and smoothed to remove loose material and to eliminate the need for large plate adjustments. For lifts compacted with the sheepsfoot roller, soil had to be removed down to the footprints over an area 8 in. wide by 40 in. long to accommodate the plate and reference arm. When the entire assembly was lowered for a test the reference feet first touched the ground, establishing the zero deflection position and activating the potentiometer. The shaft loading the plate then slid through the center bushing in the crossarm until the plate made contact with the ground and came to rest under a prescribed seating load (200 lb). The load cycle was then activated and the plate loaded at a rate of 500 lb/sec to a maximum load of 4500 lb or a maximum sinkage of 1 in.

The plot of load vs displacement was obtained beginning at the time when the reference feet first touched the ground. If the plate sinkage exceeded 1 in. during any stage of the loading, a limit switch was contacted causing the load to be automatically removed and the plate retracted. Total time for a test including surface preparation was approximately 3 min, except that when the sheepsfoot roller was used the time was doubled.

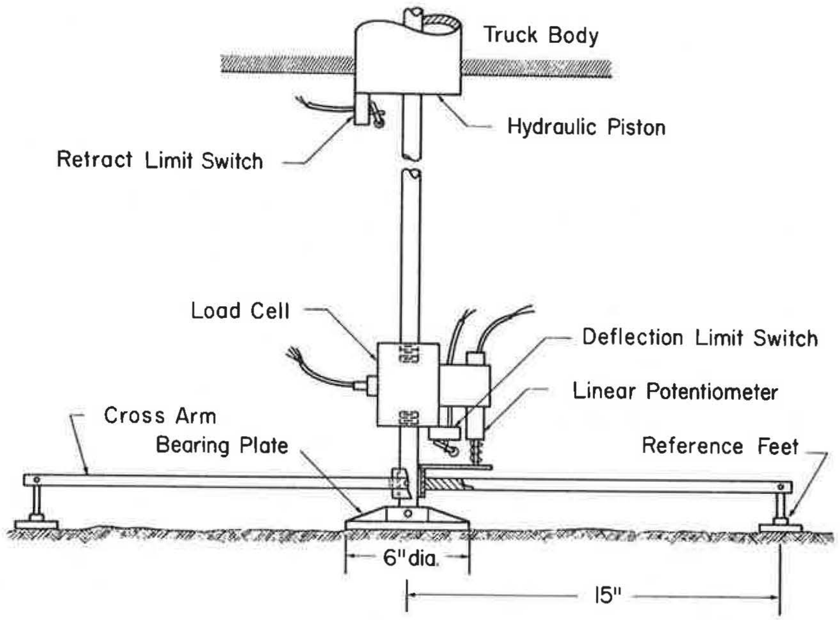

Figure 3. Bearing plate mechanism.

Although this apparatus was generally reliable, under continual heavy use the power requirements were beyond the capacity of the battery. However, this problem arose only two or three times during the testing program. On those occasions plate tests were performed by coupling a hand hydraulic pump into the system. Because the plate is sensitive to surface conditions, considerable variability will be introduced unless a rapid means of preparing the surface can be developed. This test can be used for all soils and base course materials.

California Bearing Ratio

The CBR test is a penetration test having the function of measuring the soil resistance to penetration prior to reaching its ultimate shearing value. The CBR is defined as a ratio, in percent, of the load at 0.1 or 0.2 in. penetration in the material being tested to the load at the same penetration in a standard well-graded crushed limestone.

The apparatus fabricated for performance of field CBR tests included a piston, surcharge weights, and loading device. The piston had an end area of 3 sq in. (1.95 in. in diameter) and was 5 in. long to enable it to pass through the surcharge weights and penetrate the soil. A 6-in. diameter plate was fixed to the top end of the penetration piston to permit it to be clamped to the bearing plate apparatus. The surcharge consisted of 3 annular steel rings 10 in. in outside diameter and $2\frac{1}{8}$ in. inside diameter by 1 in. thick. These weights produced a surcharge pressure per unit area equivalent to a 20-lb surcharge for the laboratory CBR test.

The loading device consisted of a hydraulic hand pump connected to the bearing plate hydraulic system. The hand pump allowed load to be applied at different rates and permitted control on the strain rate and maximum load applied. However, it was difficult to apply the rate of penetration slowly enough to equal the standard rate. The applied load and distance of penetration were plotted on an x-y recorder.

Because the test was accomplished with the aid of a hydraulic hand pump, there was virtually no possible source of trouble and no downtime was experienced. The total time required for a CBR test was approximately 3 min. Preparation was limited to leveling the soil surface over an area large enough to accommodate the surcharge weights. No sand cushion or seating cap was used for either the piston or the weights in order to minimize the total measurement time. This may have affected the CBR values. This test also requires the development of a rapid means of surface preparation, if it is to be useful for checking compaction.

Seismograph

The seismograph is used to determine the propagation velocity of small disturbances through soil. The test is a rapid and nondestructive one which requires little or no soil preparation. Theoretically the seismic wave velocity is related to the density and elastic properties of the soil. The purpose of using the seismic test in this study was to determine if the wave velocity is suitable for indicating changes in compaction through changes in these other properties.

The seismograph used in the field tests was a model MD-3 provided by Minnetech Labs, Inc. It was modified to permit measurements over horizontal distances of 6 to 24 in. in the soil. This is much less than the distances normally considered in seismic surveying. The majority of commercial instruments are not suitable for this close-in work because they do not adequately measure the short travel times involved. The selected horizontal distances are based on the thickness of the layers being tested. As a rule the distance should not exceed three times the layer thickness so that higher velocity underlying layers will not influence the measurement in the top layer being tested.

The only preparation was to remove loose soil. The total test time was approximately 3 min. Because the seismograph used was a developmental model some difficulties were encountered, but these were gradually ironed out during the program. This technique may be used with all soil and surface conditions.

The apparatus consists of four essential parts: (a) a means to induce a seismic wave into the soil, (b) a triggering device, (c) a sensing device, and (d) a time counter. The wave was induced by striking a metal bar or spike held firmly on the ground surface with a hammer. The triggering circuit was arranged so that the circuit was closed (starting the counter) when the hammer made contact with the bar or spike. A geophone anchored to the soil by either a flat plate or spike served as the sensing device.

A set of five readings was taken at each of five 3-in. intervals over horizontal distances from 18 in. to 6 in. between the hammer and the geophone on 6-in. lifts, and from 24 in. to 12 in. on lifts of 12 in. or greater in thickness. The bar was used as the wave-inducing device exclusively on base course materials. Either the bar or spike was used on soil lifts depending on surface conditions produced by the compactor. The bar was generally used if the compactor produced a reasonably smooth, firm surface condition. Wave velocity was determined as the slope of the best fit straight line to a plot of average travel time vs distance from the geophone. In general, there was a consistent relationship between the five sets of readings at any measurement location.

Seismic measurements indicate if layering occurs when the density of the layers increases with depth. In such cases over short distances the earliest wave arrival time will be through the upper layer, but beyond a distance determined by the upper layer thickness, the earliest arrival time will be through the lower denser layer where the wave velocity is greater. However, there will be no indication of layering in cases where density decreases with depth, because the earliest wave arrival will always be through the upper more dense layer.

Other Tests

Sand cone tests were performed on approximately half of the 256 test sections. They were performed as part of the final lift inspection and were taken at the immediate location of the final portable nuclear measurements. The test was performed as prescribed by ASTM standard D 1556-64. Ottawa sand was used and the apparatus calibrated frequently as outlined in Note 4 of the standard. Moisture determination was made based on oven-drying of the entire sand cone sample for a minimum of 24 hours.

Immediately prior to initiating compaction, moisture samples were taken from a randomly selected location within each test section. Samples of approximately 300 grams each were taken at 3-in. intervals through the lift thickness.

TABLE 1

RELATIVE SIGNIFICANCE OF INDEPENDENT VARIABLES ON SOIL MEASUREMENTS

Measurement	Avg. or Shaved	Individual Effects					Joint Effects										
		M	T	S	C	E	MT	MS	MC	ME	TS	TC	TE	SC	SE	CE	Cov
Final w	S	1		1													na
Initial w		1		1	6			4									na
Proctor w		1		1				2									na
Portable nuclear γ_w	S	1		1		1		7		2			7			6	
Portable nuclear γ_w	A	1		1		1				1					6	5	1
Portable nuclear w_d	S	1		1						1						5	1
Portable nuclear w_d	A	1		1	6	1		5		3							1
Road Logger γ_w	S	1	2	1	6	6				5			5			5	na
Road Logger w_d	S	1		1						4						3	na
Sand cone γ_w	S	1		1			6	4	5								na
Pen. resistance	S	1		2		2		2		4			6				na
Pen. resistance	A	1		1		1		1		4			1		7		na
Plate load	S	1				4			5	4					7		na
Plate load	A	1	3			1		1		2						5	na
CBR	S	1	5					7					4				na
Seismic velocity	S	5	4					6									6
Seismic velocity	S	6	4					7									na
Seismic velocity	A	1		6		6		1	6	1							6

Error Probability (%): 1 = 0.1, 2 = 0.5, 3 = 1.0, 4 = 2.5, 5 = 5.0, 6 = 10.0, 7 = close to 10.0.
γ_w = wet density, w_d = moisture density, w = moisture content (%).

At the same time, samples were taken at another location in each section for T-180 compaction tests. Two 300-gram samples also were taken from each of these compaction samples for moisture determination. Additional moisture samples were taken at the final inspection of each test lift in those sections in which sand cone tests were not performed. These samples were approximately 5 lb each and were obtained at the immediate location of the last portable nuclear measurement. All moisture samples were oven-dried for a minimum of 24 hours.

EVALUATION OF METHODS OF MEASUREMENT

In assessing the relative merits of the various methods of measuring properties of compacted soil several factors will be considered:

1. Ability to detect the effect of the independent variables, i.e., moisture content, lift thickness, soil type, compaction equipment, compactive effort, and number of coverages on the measurements;
2. Range and variability of the measured properties; and
3. Correlation between different methods of measuring a given property and between the different properties.

The test series consisted of 64 lifts with 4 moisture zones each or a total of 256 test sections. Since measurements were made at one location in each test section after compaction coverages 2, 4, 8 and 16, a possible total of 1024 observations could be made with each instrument during the compaction process. An additional 256 observations were possible on the shaved lift after 16 coverages of the roller. The data for each measurement were reduced on an electronic digital computer and analyzed individually using statistical analysis of variance procedures. The analysis yielded the following information: (a) the independent variables (assignable causes) which had a significant effect on the measured quantities, (b) the variability (chance causes) associated with the measurement, (c) estimated mean value of the measurement for each level of the independent variables and their two-way interactions, and (d) coefficients for computation of the characteristic growth curves for each level of the independent variables and their interactions. Correlation between measurements can be obtained through comparison of the generated expected mean values.

Significance of Independent Variables

Two considerations regarding the effect of the independent variables on the measurements are: (a) does the property being measured vary significantly with respect to the independent variables, and (b) can each particular instrument sense the real changes which occur? To answer these questions, Table 1 summarizes for each measurement those variables and their joint interactions which the statistical analysis indicated had a significant effect. The independent variables are M = moisture level, T = lift thickness, S = soil type, C = compactive effort, and E = compaction equipment. Combinations of any two letters indicate joint effects; "Cov" indicates covariate.

Each measurement is subdivided into two groups where appropriate to represent the average of the growth values (A) for coverages 2, 4, 8 and 16 and the values for the shaved lifts (S) after compaction. The confidence level of significance is expressed in terms of the probability that the indication of significance is a chance occurrence. The probabilities selected range from 0.1 percent to 10 percent. In all cases not marked the probability exceeded 10 percent.

Table 1 shows that initial, Proctor, and final oven-dried moisture contents were essentially functions of only moisture level, soil type and their interaction. With one exception, all other independent variables showed no significant correlation (at or close to the 90 percent confidence level) with the oven-dried moisture determinations. It was desired that correlations exist only for moisture level, soil type, and joint moisture-soil effects, the latter because the planned moisture increments were different for the clay soil than for the other soils. Correlation between the moisture content measurements and any other variable would indicate an unwanted bias in the test plan, which would effect the analysis of all other measurements. A possible bias is indicated by the initial moisture measurement with respect to compactive effort. It is significant, however, only at the 90 percent confidence level, i.e., such a correlation may be expected to occur by chance approximately 1 time in 10. No correlation is seen between compactive effort and either the Proctor or final moisture measurements. Based on these observations, it is concluded that the test plan was adequately randomized.

Joint effects of thickness with soil and compactive effort, and joint effects of soil with compactive effort are not significant with respect to any measurement. Hence, within the detection ability of any measurement, the effects of the independent variables thickness, soil and compactive effort are independent of each other; for example, the effect of soil type on any measurement holds for both values of thickness and both values of compactive effort.

The column titled covariate applies only to the portable nuclear and seismic instruments. Two different portable nuclear instruments were used for the tests and three different switch types were employed with the seismic unit. The introduction of a covariate term in the analysis of the data was made to permit determination of effect on the results caused by these changes. The analysis adjusted the data to remove such effects if they existed.

The instrument effect for the portable nuclear moisture density measurements was highly significant. An instrument effect also showed up for the wet density growth measurements. This effect was not evident for the wet density measurements on the shaved lifts. The level of significance makes it highly unlikely that only a chance occurrence is being observed in the growth measurements with respect to the covariate effect. It is believed most likely that the measurements on the sheepsfoot-compacted lifts are producing the covariate significance. The considerable effect of these lifts on the portable nuclear measurements will be demonstrated later.

A covariate effect was also evidenced with the seismic instrument, but at a much lower level of significance. Because of this, the seismic data were also analyzed, disregarding the covariate effect for the shaved lift measurements. There was no change in those variables which influenced the measurement and only slight change in the significance levels of the effects.

Other observations which should be made from Table 1 are:

1. The effect of moisture level can be detected by all measurements and, with the exception of the seismic measurements, the level of significance makes it highly unlikely that detected effects occur by chance.

2. The Road Logger, bearing plate (growth), CBR and seismic instruments detected changes in measured properties with changes in lift thickness, while the portable nuclear gage, penetrometer and sand cone detected no difference.

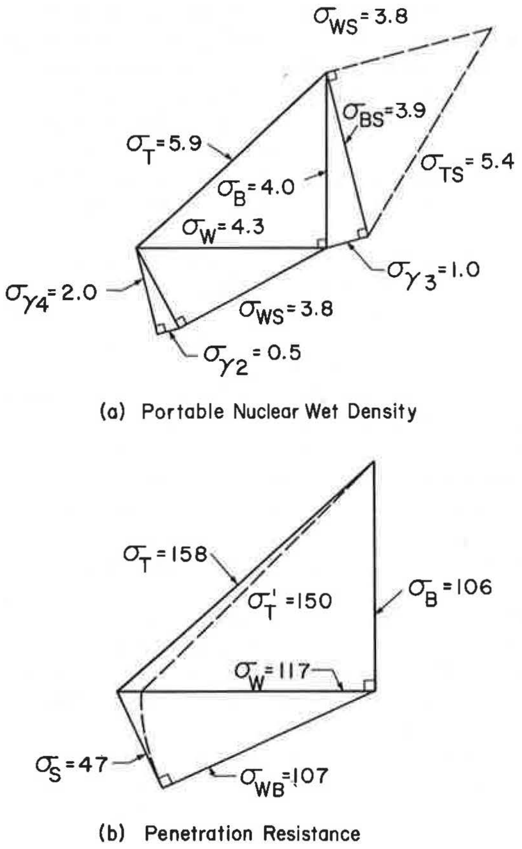

Figure 4. Standard deviation vector diagrams.

3. The bearing plate, CBR and seismic measurements were rather insensitive to soil type, while soil type was highly significant with respect to the other measurements.

4. There was little detectable effect of compactive effort alone, but the joint effect with equipment indicates that the influence of changes in compactive effort depends on the compaction equipment being used.

5. The type of compactor used resulted in detectable differences in wet density, penetration resistance, and bearing strength.

6. Only interrelationships of moisture-soil, moisture-equipment, and thickness-equipment appear to be of measurable consequence.

Variability of Measured Properties

The statistical analysis which determined the effects of the independent variables on the measured values predicted the best estimated mean values associated with each level of the variables. These are equal to the averages of the actual measurements only when no data are missing from any test section. To each measurement a standard deviation can be assigned which defines the variability of that individual observation or measurement. The analysis performed predicted this standard deviation based on the residual variance after the effects of the selected independent variables had been removed. Each level of each variable may have a different variability associated with it, but it is not possible to determine these differences with the particular test plan and analysis used; therefore, the variance is considered to be constant for all effects. The error produced by this assumption is believed to be of second order.

The analysis computes the total variance (σ_T^2) of each measurement and permits it to be separated into the two components corresponding to "within-lift" (σ_W^2) and "between-lift" (σ_B^2) variability. These variances are related by

$$\sigma_T^2 = \sigma_W^2 + \sigma_B^2$$

The within-lift variance (σ_W^2) represents the variability about the mean of each test section caused by such factors as the variation in moisture content from the nominal values, the point-to-point variation of properties within each test section, and measurement errors such as seating effects for the neclear gage. The variance σ_W^2 is comprised of within-test section (σ_S^2) and between-test section (σ_{WB}^2) components, but the experiments conducted only provide a direct estimate of σ_W^2. The value σ_B^2 includes the effects of changes in environmental conditions, lift foundation conditions, and heterogeneity of the soil stockpile. The value σ_T^2 includes the errors contributing to σ_W^2 and σ_B^2.

Figure 4 shows components of error in terms of standard deviation for the portable nuclear gage and penetrometer measurements. In Figure 4a the large triangle represents the total error σ_T and its two components σ_W and σ_B. The errors σ_W and

σ_B are in turn divided into two components, one representing the estimated measurement error and the other representing the density variation in the soil itself. The components of σ_W are shown in the lower triangles in Figure 4. They include measurement errors $\sigma_{\gamma 2}$ caused by random backscatter[1] and $\sigma_{\gamma 4}$ caused by seating effects[2]. The vector sum of these is subtracted from σ_W to obtain σ_{WS}, the within-lift soil density error. The between-lift component (σ_B) is shown divided into the maximum measurement error due to temperature ($\sigma_{\gamma 3}$) and the remaining portion which is the between-lift soil density error (σ_{BS}). The total soil density error (σ_{TS}), which is the vector sum of σ_{WS} and σ_{BS}, is shown as a dashed line.

The values σ_{BS} and σ_{WS} are approximately the same. A major cause of this variability is believed to be difficulty in moisture control. It is evident that σ_{TS} does not represent a normally distributed error, since normal distribution theory indicates that 5 percent of the data will fall beyond $\pm 2\sigma$ or ± 10.8 pcf. The distribution of σ_{TS} is probably peaked; however, the possibility also exists that unknown effects including three-way and higher interactions between independent variables may have caused these computed standard deviations to appear higher than they actually are.

Also, the density measurement errors due to temperature ($\sigma_{\gamma 3}$) and backscatter ($\sigma_{\gamma 2}$) were small in comparison to the other errors with which they are associated. Their elimination by modification of the measurement procedures would produce no significant improvement in the estimation of the soil density in these experiments. The seating error $\sigma_{\gamma 4}$ is important with respect to σ_W, but will not substantially reduce σ_T.

Figure 4b shows the extent to which replicate measurements would have improved the ability to detect changes in penetration resistance. Two measurements of penetration resistance were made, each at a different point in every test section after the 16th compactor coverage. This permits the σ_W to be broken down into within-section (σ_S) and between-section (σ_{WB}) components. It may be seen that any replicating of measurements would have resulted in little gain. Assume that enough replicates (repeated measurements within a test section) were made to reduce $\sigma_S/\sqrt{N}$ to a negligible amount. Then σ_W would become equal to σ_{WB}, a reduction of 10 lb or 8.5 percent, and σ_T would become equal to σ_T', a reduction of only 8 lb or 5.1 percent.

The values shown in Figure 4 represent error in single observations. If more than one observation is made, then the appropriate error component will be divided by the square root of that number of observations. It is evident that the best procedure to use is that which replicates the measurement's largest error component. According to Figure 4a, the most suitable procedure to use with the backscatter instrument would be to obtain a series of readings at random locations in each test section, making only one 1-minute count at each location; i.e., eliminate any gage rotation and duplicate counts at each location and use the time saved to include as many locations as possible. However, information from the penetrometer measurements indicates that even this would probably not have substantially reduced the total measurement error and therefore the cost of the additional measurements would not have been justified. Only replication of test conditions by using new lifts would have improved on the accuracy of the experiments.

To compare the various methods of measurement on the basis of measurement error, it is necessary to reduce all errors to a common (normalized) form. The range of values of the properties measured is believed to be the most relevant base for this conversion. This range was determined as the difference between the minimum and maximum values of each measured property for all combinations of the independent variables. Table 2 gives the standard deviation, range, mean value and ratio of standard deviation to range for each measurement.

The within-lift standard deviation is probably a reasonable estimate of the variability of properties within a compacted embankment. According to Table 2, the variation in moisture content had a standard deviation of 1.2 percent. Associated with this

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[1]Computed assuming standard deviation of counts is $1/\sqrt{N}$, where N is the number of counts.

[2]Estimated based on laboratory studies.

TABLE 2

MEAN, RANGE AND VARIABILITY OF MEASURED PROPERTIES

Measurement	Dimension	Mean μ	Range R	Standard Deviation σ_W	σ_B	σ_T	σ_W/R	σ_T/R
Final w	%	11.7	11.4	1.2	1.0	1.6	0.105	0.140
Portable nuclear γ_w	pcf	126.7	27.6	4.3	4.0	5.9	0.156	0.214
Portable nuclear w_d	pcf	11.5	9.7	1.4	1.2	1.9	0.144	0.196
Road Logger γ_w	pcf	128.5	32.0	3.6	4.7	5.9	0.113	0.184
Road Logger w_d	pcf	13.8	12.1	1.2	1.6	2.0	0.099	0.165
Pen. resistance	lb	364	529	117	106	158	0.222	0.299
Plate load	lb	1720	1814	677	445	810	0.373	0.446
CBR	%	15	26.6	7.3	8.8	11.4	0.274	0.277
Seismic velocity	fps	1192	890	344	119	364	0.387	0.410

were standard deviations of about 4 pcf for wet density, 117 lb for penetration resistance, 677 lb for plate load, 344 fps for seismic velocity and 7.3 percent for field CBR.

The ratio of variability to range is one valid criterion for comparing methods of measurement. The lower the ratio, the greater is the ability to detect changes in properties for a given number of measurement observations. This method of ranking would rate measurement of density best, CBR second, penetration resistance third, and seismic velocity and plate load about the same as last. This ranking should not be considered as absolute, however. The ratio of variability to range can be reduced by $1/\sqrt{N}$ (N = number of observations). Thus, if time and cost considerations permit more measurements of one type to be made than the others, then considerable change in ranking can occur.

One advantage of the continuous logging capability of the Road Logger is also seen from these ratios. The Road Logger measurements provided an estimate of the average value for each section. The effect of this is evidenced in the σ_W/R ratio which is much smaller for the Road Logger than for the portable nuclear instrument. The Road Logger should be able to detect more effects of the test variables than the portable nuclear instruments. The data in Table 1 indicate that this was in fact the case.

CORRELATION OF MEASUREMENT METHODS

The preceding discussion dealt with the various property measurements individually. The remaining discussion considers the correlation between measurements both of the same property, such as nuclear vs oven-dried moisture content, and different properties, such as penetration resistance vs density.

Moisture and Density

Oven-dried measurements of moisture content were available from sampling each test section prior to initiating compaction, from the field Proctor test (samples for Proctor tests were also taken from each test section prior to initiating compaction), and from each test section upon completion of testing on the shaved lift surface.

Of these three, it is believed that the final moisture content measurements were the most representative of lift moisture content during compaction. These samples were approximately 5 lb in weight and included a normal amount of coarse material such as gravel content. The initial moisture contents were obtained from the average of two to four 300-gram samples, and the Proctor moisture contents from the average of two 300-gram samples. These samples probably did not contain a normal amount of coarse material, because the larger size particles were excluded.

Moisture content determinations were also obtained from the portable nuclear and Road Logger measurements. Figure 5 compares all moisture content (%) measurements with respect to the final oven-dried measurements. The initial, Proctor, and Road Logger measurements correlate well, but are generally slightly higher in magnitude than the final measurements, with the Road Logger moistures tending to be slightly low at the high end of the range. The portable nuclear measurements are

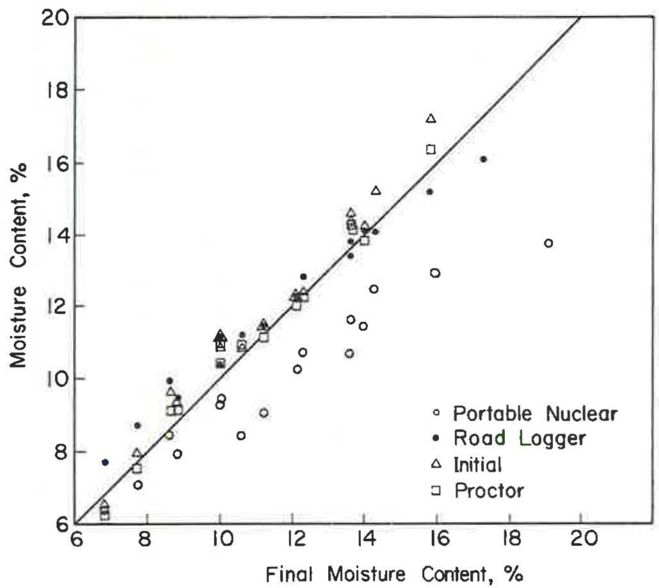

Figure 5. Comparison of oven-dried and nuclear measurements of moisture content; data averaged for each combination of moisture level and soil type.

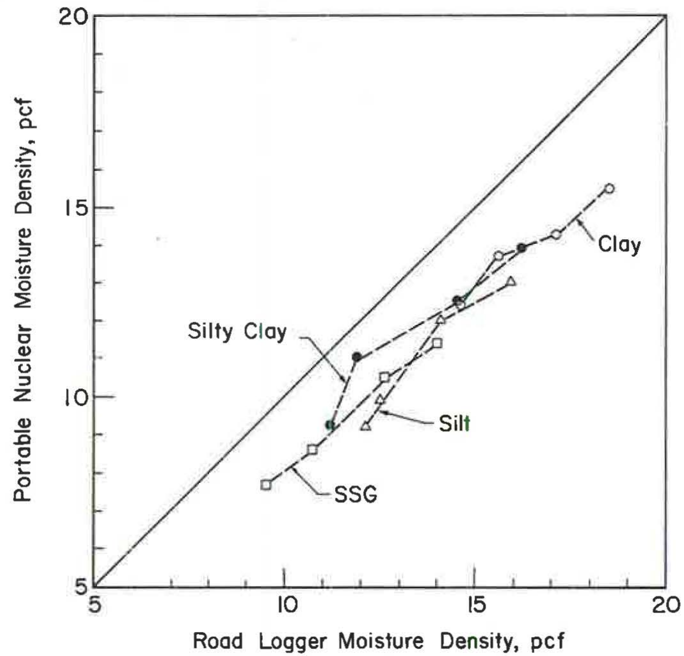

Figure 6. Comparison of portable nuclear with Road Logger measurements of moisture density; data averaged for each moisture level for each soil type.

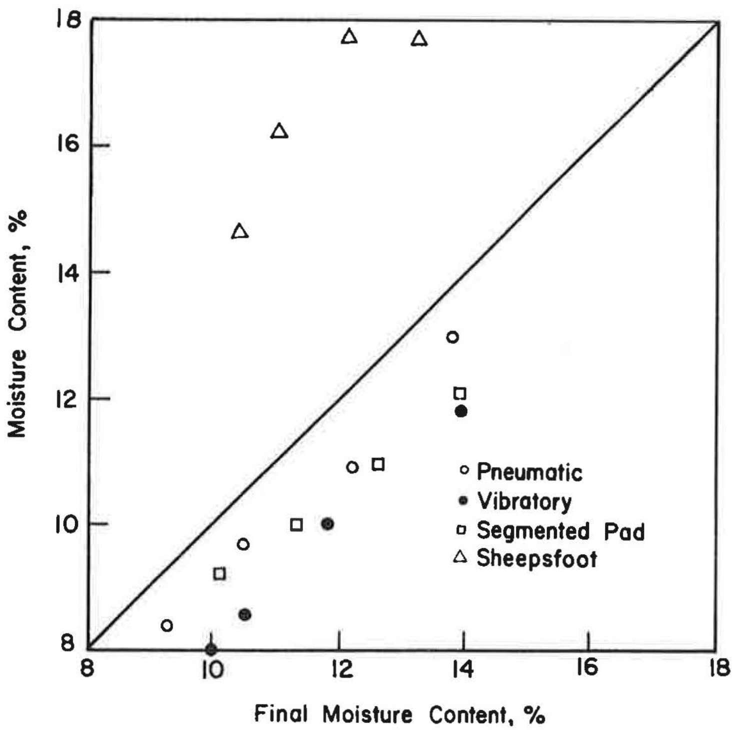

Figure 7. Comparison of average growth measurement portable nuclear moisture contents with final moisture content; data averaged for each moisture level for each equipment type.

considerably lower than the final moisture contents over the entire moisture range. This is believed to be due primarily to error in the portable nuclear measurement of moisture density rather than wet density.

Figure 6 compares Road Logger and portable nuclear measurements of moisture density. Correlation is linear and consistent for all moisture-soil combinations, but the portable nuclear measurements are all low with respect to the Road Logger measurements. The better agreement of the Road Logger with the oven-dried moisture contents suggests that it is the portable nuclear measurements which are in error.

Figure 7 shows the difficulty of obtaining accurate measurements with the portable nuclear instrument on sheepsfoot lifts during compaction. The measurements on lifts compacted with the sheepsfoot roller are considerably different from those for all other rollers. For the sheepsfoot lifts, the portable nuclear measurements of moisture density were too high and the measurements of wet density were slightly low. These errors are additive in the moisture content determination. There were several problems with measurements on sheepsfoot-compacted lifts. First, it was difficult to determine just how much material should be removed for the measurement. Second, it was difficult to prepare a smooth and level surface. Finally, there were large local variations. Points which had been directly under a foot were hard and firm, while a few inches away the material would be much looser. This was particularly true during the early stages of compaction.

Figures 8 and 9 compare portable nuclear and Road Logger shaved lift measurements of wet density for data averaged over moisture-soil and moisture-equipment combinations, respectively. With the exception of the curves for the silty clay soil and the segmented pad roller, both sets of data would be fairly well represented by a line parallel to the 45-deg line (1 to 1 correlation), with the portable nuclear reading about 2 lb/cu ft lower than that of the Road Logger. There is, however, no apparent reason why either silty clay soil or segmented pad roller should cause a relative

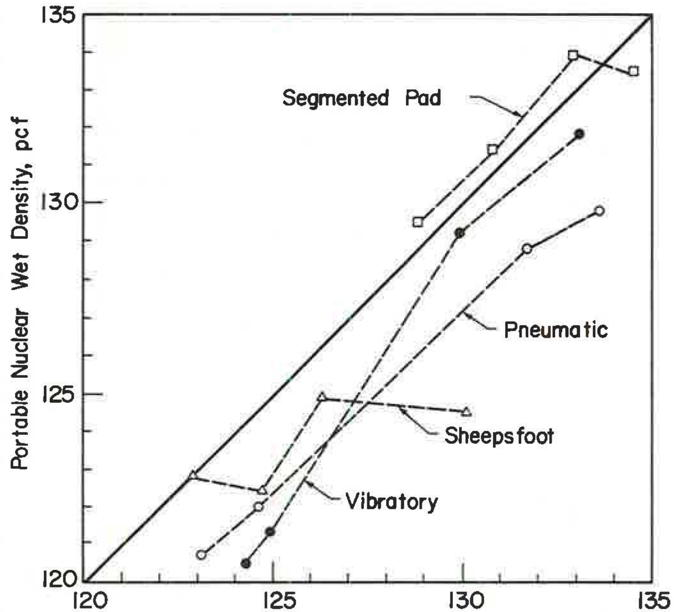

Figure 8. Comparison of portable nuclear with Road Logger measurements of wet density; data averaged for each moisture level for each equipment type.

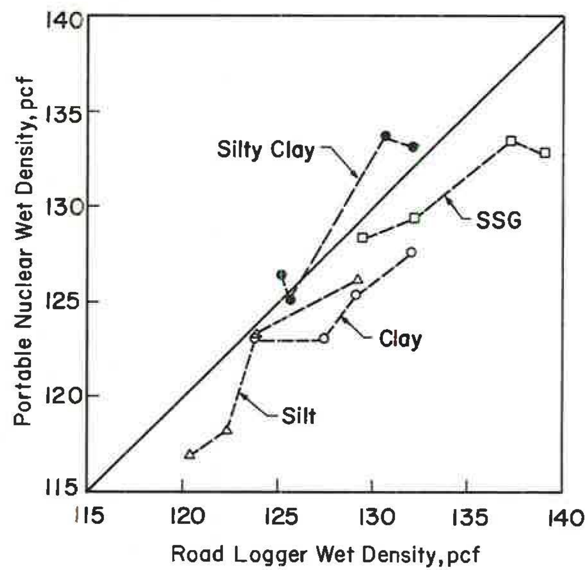

Figure 9. Comparison of portable nuclear with Road Logger measurements of wet density; data averaged for each moisture level for each soil type.

influence on the nuclear measurements. As such, the best overall relationship would be expressed by straight lines skewed to 45 deg. The general predominance of the points below the 45-deg line may to some extent be due to missing data caused by equipment malfunctions.

The average wet densities, as determined from the analysis of variance, for all measurements with the two nuclear methods agree within 2 pcf—128.5 pcf for the Road Logger vs 126.7 pcf for the portable nuclear instrument. It is believed that correlation between these two methods would have been better had there not been missing data points. Of the 256 shaved lift observations, 232 were obtained with the portable nuclear instrument and 212 with the Road Logger. Of these, 208 observations were in common, i.e., performed on the same test section. The means of these common observations were within 1 pcf—127.7 pcf for the Road Logger vs 126.6 pcf for the portable nuclear instrument.

Since each of the above means are based on more than 200 observations, it is safe to assume that the means are normally distributed. Estimates of variance of the means should also be quite good, based on the large number of observations. A statistical test may be applied to determine if real differences exist between the Road Logger and portable nuclear means. The critical region for the test is chosen to consist of the two equal tails of the distribution of the two means $(\bar{\gamma}_w)_{RL}$ and $(\bar{\gamma}_w)_{PN}$. If the usual critical region size of 0.05 is selected, then the differences in the means determined from the analysis of variance come near being significant while the differences of means determined for common observations are not significant. Thus, within the variability associated with the means, the differences are small enough to consider the means as the same.

Valid comparisons with sand cone measurements are quite limited. More than 50 percent missing data exist with the sand cone measurements. Consequently, the results given by the analysis must be highly qualified. However, both the analysis and means of common observations indicate the sand cone wet densities were about 4 pcf less than the nuclear measurements. It is believed that the nuclear measurements are the more correct. The main problem or source of error in sand cone tests is believed to be the low relative density of the sand cone calibration—approximately 96 pcf. A slight disturbance could easily increase the density to 100 pcf, introducing a 4 percent error.

Soil Strength and Seismic Velocity

Comparison of seismic velocity, plate bearing load, penetration resistance and CBR values with wet density for the four moisture levels is made in Figure 10. All data are averaged for each of the four moisture levels, thus each point represents approximately 64 observations. Moisture level increases from left to right in the figure. The general shape of the curves relating measurements of soil strength, i.e., plate bearing load, penetration resistance, and CBR, with wet density are quite similar. The strength values decrease with increase in wet density, which in turn corresponds to increase in moisture level. These curves will bend back toward the origin at higher moisture levels. Seismic wave velocity appears to reach a peak value within the range of wet density encountered. Its behavior is quite different from that of strength.

Figure 11 compares penetration resistance and plate bearing load with CBR, averaging independently over moisture level and equipment type. The relationships changed with soil type. The best correlation was between plate load and CBR. The only poor feature of the correlations is the low value of penetration resistance for the pneumatic compactor (P) measurements. While no apparent reason for this discrepancy can be given, the effect is considered real, since it represents the mean of 64 observations.

Correlation of seismic velocity with strength measurements is not presented because, in general, it was found to be quite poor. This is evident in Figure 10 where seismic velocity increased and then decreased with increasing wet density while all strength measurements continued to decrease.

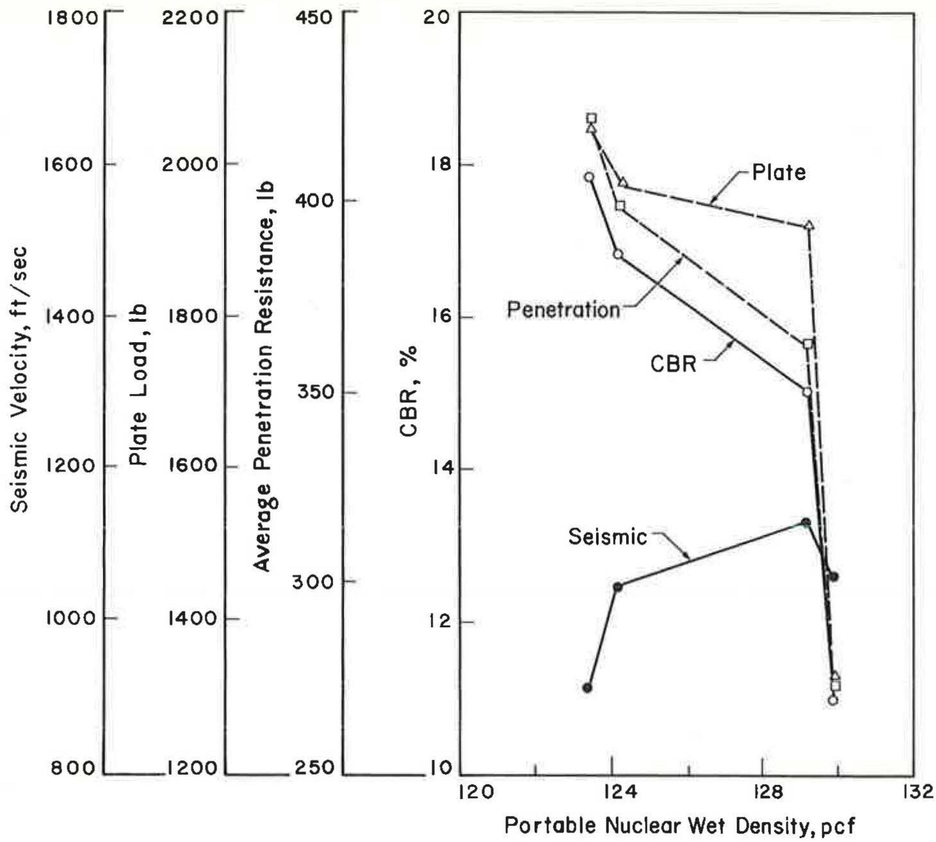

Figure 10. Comparison of seismic velocity and soil strength measurements with wet density; data averaged for each moisture level.

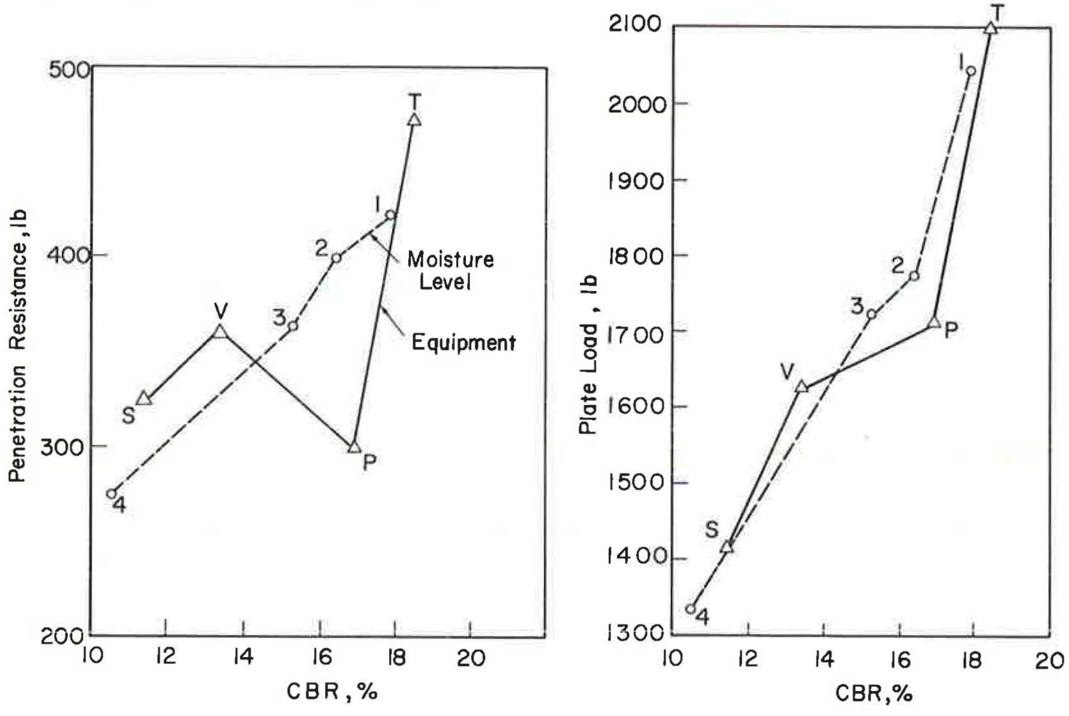

Figure 11. Comparison of penetration resistance and plate load with CBR; data averaged for each moisture level and equipment type.

SUMMARY AND CONCLUSIONS

The field compaction study included the evaluation of several methods of measuring the properties of compacted soils. The large variability associated with individual measurements restricted the study of the data with respect to the independent variables to the predicted averages of many observations. While this requires that conclusions drawn be rather general in nature, several important statements can be made with a high degree of certainty.

The test plan appeared to be the most efficient for accomplishing the intended scope within the funding allocated. Replicate measurements within test sections would not have materially improved the ability of the data to detect the effects of the independent variables, and such replication would have increased the cost of the field operation by possibly 50 percent. Replication of lifts or significantly improved moisture control would have at least doubled the cost.

Variability of properties in a compacted lift are much larger than generally assumed or admitted. Single observations of any of the properties measured would seem to be of little value in assessing the adequacy of compaction. Either many properly selected measurements are required or considerable judgment based on observed construction procedures and compactor performance is necessary. Little reliable variability data are available from actual construction jobs. One of the most important contributions of further field studies of compaction would be the gathering of such information.

All measurement of soil properties included in the field tests were capable of detecting the changes occurring with additional roller coverages. The effects of the independent variables on the property of wet density were among the easiest to detect because the ratio of variability to range was smallest for the nuclear instruments. However, if multiple observation of other measurements could be as easily obtained as single nuclear measurements, then the detectability with the former could be as good because the relative variance would be determined by the ratio of the number of observations with the two methods.

On the whole, the portable nuclear instrument gave the same measurement of wet density as the Road Logger, even though the Road Logger used much higher strength nuclear sources and thus was less sensitive to soil composition and surface conditions and provided a greater depth of averaging. Calibration of the portable nuclear gages still appears to be a problem needing study for both density and moisture measurements. General experience indicates that the operator cannot assume that these gages will work properly and that the supplied calibration curves will be correct. A thorough check-out is required with every new gage. Standard operation procedures for nuclear measurements on soil are badly needed. If properly functioning and properly used, the portable nuclear gages can give useful results. Considering actual variability of the measured properties in the field in relation to the instrument variability, it is not considered worthwhile to obtain more than one 1-min count at any location or even to rotate the gage and repeat the reading at the same location. All replicates should be obtained at different locations. Rapid and accurate methods for preparing the soil surface would be a significant aid in improving and speeding up measurements with these gages.

The Road Logger appears to provide good readings of both wet density and moisture density; therefore, the dry density and moisture content calculations should be reliable. The logging capability of the Road Logger makes it the only device which provides information on uniformity of compaction over large areas easily and quickly. Such information is as important as the numerical values obtained. This advantage was illustrated in this program by the lower ratio of within-lift variability to range, and by the ability to detect changes in wet density with some independent variables which were not apparent with the portable nuclear measurements. However, the Road Logger is restricted to use on rather uniform surface conditions. It could not, for example, be used on the type of surface condition created by a sheepsfoot compactor.

On the whole the same general type of information was obtained with the penetrometer, plate, and CBR soil strength measurements. The penetrometer has four important advantages with respect to all other measurements: (a) it requires almost no surface preparation, (b) it is free of operator error, (c) it is extremely rapid, and (d) it permits an examination of vertical variations in the lift. It cannot be used with base course materials, however. The bearing plate, to be useful as a control device, requires the establishment of a technique for rapid surface preparation. It can be used on all soil conditions. The CBR is not recommended as a control device, but was included in the test program for comparison purposes only. For any soil strength measurement to be used as a compaction control device, some measure of moisture content will also be required.

It was not evident from these tests how the seismic device could be applied to compaction control. A better understanding of the influence of the properties of compacted soils on its measurements is needed to properly assess its role. The method was rapid, nondestructive, required little soil preparation and was able to detect clearly changes with roller coverage.

One of the most useful measurements to be able to perform rapidly and accurately is moisture content (%). This could be the most valuable inspection measurement for obtaining the desired compaction end result. Finally, the experience on this study suggests that the use of more sophisticated measuring apparatus which can permit more rapid and thorough inspection and control, even if requiring greater capital investment of operating cost, is entirely justified by the benefits to be gained in improved performance, lower maintenance costs, and very likely more efficient construction procedures.

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REFERENCES

1. Hampton, Delon, and Selig, E. T. Field Study of Soil Compaction. Presented at the 46th Annual Meeting and published in this RECORD.
2. Selig, E. T., and Truesdale, W. B. Properties of Field Compacted Soils. Presented at the 46th Annual Meeting and published in this RECORD.

Properties of Field Compacted Soils

E. T. SELIG, Manager, Soil Mechanics, and W. B. TRUESDALE, Research Engineer, IIT Research Institute, Chicago

Soil compaction tests were conducted in the field by constructing test sections of soil in single lifts on a prepared foundation using a variety of commercial rollers. The test results were obtained using the following specific independent variables: (a) four subgrade soils, A-6(13), A-6(9), A-4(1) and A-4(8); (b) four moisture contents for each soil ranging from dry to wet of optimum; (c) two lift thicknesses, 6 and 12 in.; (d) four rollers, sheepsfoot, pneumatic tire, vibratory smooth-wheel, and segmented pad, at two levels of effort for each roller; and (e) roller coverages up to 16. Measurements were made of the strength, stiffness and density of the soil using a variety of techniques. A full factorial experiment consisting of 256 test sections to represent all combinations of these selected variables was designed to detect, using analysis of variance techniques, the effects of the variables on the measured soil properties, taking into account the large variability existing in the field. The results describe the measured CBR, penetration resistance, bearing stiffness, seismic velocity and density, and show how they were affected by the test variables. The CBR and density are also compared with the values obtained using standard laboratory tests.

•A research program was undertaken to investigate the properties of field compacted soils and the factors involved in construction which influence these properties. Tests were carried out in the field in an attempt to simulate many of the environmental and operational conditions encountered in construction. Details of the test plan and procedures are described in a companion paper in this RECORD (1). A general familiarity with that paper will be assumed. The methods of measurement are evaluated in another companion paper (2).

The results discussed in this paper were obtained from the statistical analysis of the principal series of tests on the subgrade soils. These provided 256 test sections combining the following variables:

1. Four soils—moderately plastic clay, silty clay, silty sand and gravel, and silt.
2. Lift thicknesses of 6 and 12 in.
3. Four moisture contents bracketing the laboratory standard and modified Proctor optimums.
4. Four compactors—intermediate pneumatic, intermediate vibratory, segmented pad and self-propelled sheepsfoot.
5. Two levels of compactive effort for each roller.
6. Soil preparation by a pulverizing mixer.

Standard compaction tests were conducted on each of the four soils in the laboratory, and the modified Proctor tests were repeated in the field using samples taken from the prepared lifts. In both cases unsoaked CBR tests were performed on the compacted specimens.

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

Single lifts were prepared containing four test sections, one for each nominal moisture level. Initial moisture content was measured in each test section. After 2, 4, 8 and 16 coverages with a roller, the following measurements were made in each test section:

1. Average penetration resistance through lift.
2. Load on 6-in. diameter bearing plate causing 0. 1 in. sinkage.
3. Seismic velocity.
4. Wet density and moisture density (moisture content in lb/cu ft of water) with a backscatter nuclear instrument.
5. Wet density and moisture density with a nuclear Road Logger.

At the completion of 16 coverages the lift was stripped to approximately one-half of its thickness and the measurements repeated (the penetrometer measurements were taken before stripping). In addition, final moisture content and CBR were measured and sand cone tests performed on two of the four test sections.

This paper discusses the factors which significantly affected the measured properties, the magnitude of the effect, the nature of the growth curves, and the correlation between the different types of measurements.

EFFECT OF INDEPENDENT VARIABLES ON PROPERTIES

The independent variables and number of levels of each variable considered in the field tests were: moisture—4, lift thickness—2, soil—4, compactive effort—2, and compaction equipment—4. A statistical analysis using analysis of variance techniques was conducted to determine which of these independent variables influenced the measured properties and the magnitude of these effects. The statistical model for the analysis was constructed so that the joint effects of any two of these variables could be determined, as well as the individual effects of each alone. In addition, the variability associated with each measurement was estimated. It must be kept in mind that the measurement techniques also may have a considerable influence on the results; therefore, to the extent that these latter effects are correlated with the independent variables in a manner which cannot be predicted, then the observations will be biased. A discussion of the variability and methods of measurement is contained in a companion paper (2).

Table 1 lists the soil measurements and ranks the effects of the independent variables in order of significance. In addition to the five independent variables, the ten possible combinations of these variables are included. (The independent variables are designated as follows: M = moisture level, T = lift thickness, S = soil type, C = compactive effort, and E = compaction equipment. Combinations of any two letters indicate joint effects.) The significance is expressed in terms of a probability of error in an assumption that the given variable really affects the measurement rather than being a chance occurrence. The categories range from less than 0. 1 percent to 10 percent. Traditionally, a 5 percent limit is often selected as an upper bound, but for this analysis the limit was extended to 10 percent. Any unmarked variable exceeds that limit.

The effect on the measured values in Table 1 was based on analyses of the data from individual coverages and also the average of coverages 2, 4, 8 and 16 when such data were available. When several analyses were made for a particular measurement, the results were combined and the lowest error probability (highest significance) for each effect was shown in the table.

Moisture Content and Field Proctor Tests

The initial moisture content and sand cone moisture content, and the wet density, moisture content and CBR from the field Proctor test, were examined as a group to determine the possible existence of any unwanted bias in the test results. Only the moisture content (M) and soil (S) independent variables should be significantly correlated with these measurements, according to the test plan. Any other significant effects

TABLE 1

RELATIVE SIGNIFICANCE OF INDEPENDENT VARIABLES ON SOIL MEASUREMENTS

Measurement	Individual Effects					Joint Effects									
	M	T	S	C	E	MT	MS	MC	ME	TS	TC	TE	SC	SE	CE
Initial w(%)	1		1	6			4								
Sand cone w(%)	1		1												
Proctor w(%)	1		1				2								
Proctor γ_w	1		1				1								
Proctor CBR	1		1	4	4		1								5
Portable nuclear γ_w	1		1		1				1					6	
Portable nuclear w_d	1		1	6	1		5		1						5
Portable nuclear γ	1		1		1		5		1			6		6	6
Road logger γ_w	1	2	1	6	6				5			5			5
Road logger w_d	1		1						4						3
Plate load	1	3			1		1	5	2						5
Pen. resistance	1		1		1	6	1		4			1			
Seismic velocity	1	4	6		6		1	6	1						
Field CBR	1	5					6					4			

Note: Error Probability (%): 1 = 0.1, 2 = 0.5, 3 = 1.0, 4 = 2.5, 5 = 5.0, 6 = 10.0.
γ_w = wet density, γ_d = dry density, w = moisture content (%), w_d = moisture density.

would be present by chance and might distort the data interpretation. Table 1 shows that for each of the five measurements the effects of M and S were highly significant (at the 0.1 percent level), and the only other consistent effect present was the joint interaction of moisture and soil (MS). For the three moisture measurements no other effects were significant, except for compactive effort (C) in one case where it was just significant at the 10 percent level.

Of the five measurements only the CBR showed important deviations from the expected behavior. Compactive effort (C), equipment (E) and their joint interaction (CE) were significant at the 2.5 and 5 percent levels. The explanation for this occurrence, in view of the results with the other four measurements and an examination of the data, appears to be some chance correlation in the CBR test and not a bias in the test plan itself.

The relationships for the Proctor wet density and CBR will be discussed later. The relationship between the measured moisture contents for the three methods and the prescribed levels of M and S are shown in Figure 1. The results show a continuous increase in moisture content with moisture level. The increase with soil type for each moisture level is in the same order as the optimum moisture contents from the Proctor test. For all four soils the T-99 optimum is bracketed and, except for the clay, the T-180 optimum is also bracketed. The clay was wetter compared to its optimum than the other soils and the silt was dryer.

Nuclear Moisture and Density

The wet density (γ_w) and dry density (γ_d) measurements from the portable nuclear gage and the Road Logger on the compacted lifts were all affected by the variables M and S at the most significant (0.1 percent) level. The compaction equipment had a highly significant effect on the portable nuclear density measurements, although it was only significant at the 10 percent level for the Road Logger. Joint effects of ME, TE, SE, and CE for wet density and MS and SE for dry density were also present for the nuclear devices. The results are shown in Figure 2.

The wet density increases continuously with moisture level, hence the compaction appears to be dry of optimum on the whole for the compactive efforts used. The wet density also increases with respect to soil type in the same order as the Proctor maximums, except for the silt, which gave the lowest value instead of lying between the clay and the silty clay. The main reason for this is that the silt was compacted drier of optimum than the other soils (Fig. 1), and the clay was compacted closest to optimum

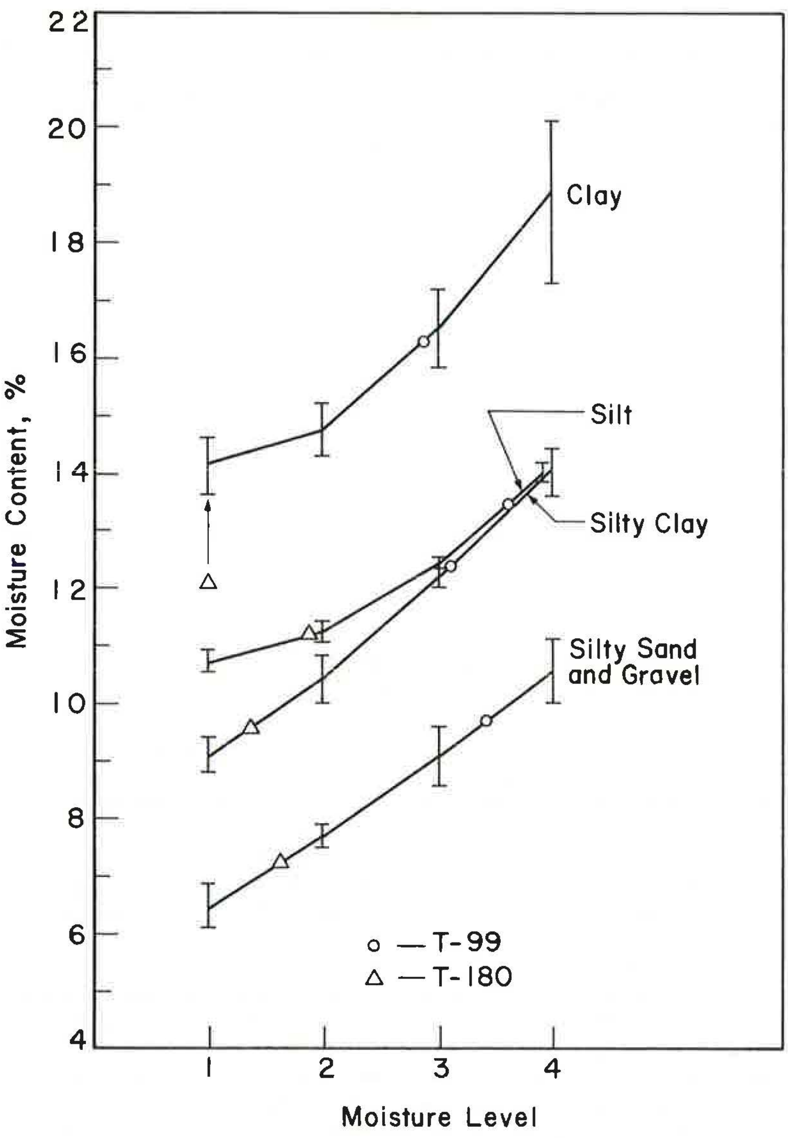


Figure 1. Moisture variation with moisture level and soil type.

for the average compactive effort. But this situation could also result if silt did not compact as effectively as the other soils. The dashed line in Figure 2 indicates the probable relationship if the moisture contents for all of the soils were the same relative to their respective optimums.

The sheepsfoot (S), pneumatic (P), and vibratory (V) rollers statistically gave about the same wet densities, and the segmented pad (T) roller gave significantly greater values on the average (Fig. 2). The values shown are averages for all soils, compactive efforts, moisture levels and lift thicknesses used in the tests. The relative relationships between the results for the four compactors will change with the specific conditions. The segmented pad roller, which gave the highest overall density, was the heaviest roller of the four, even at its lowest compactive effort. In addition, the results with the sheepsfoot roller are undoubtedly influenced adversely by using single-lift test sections.

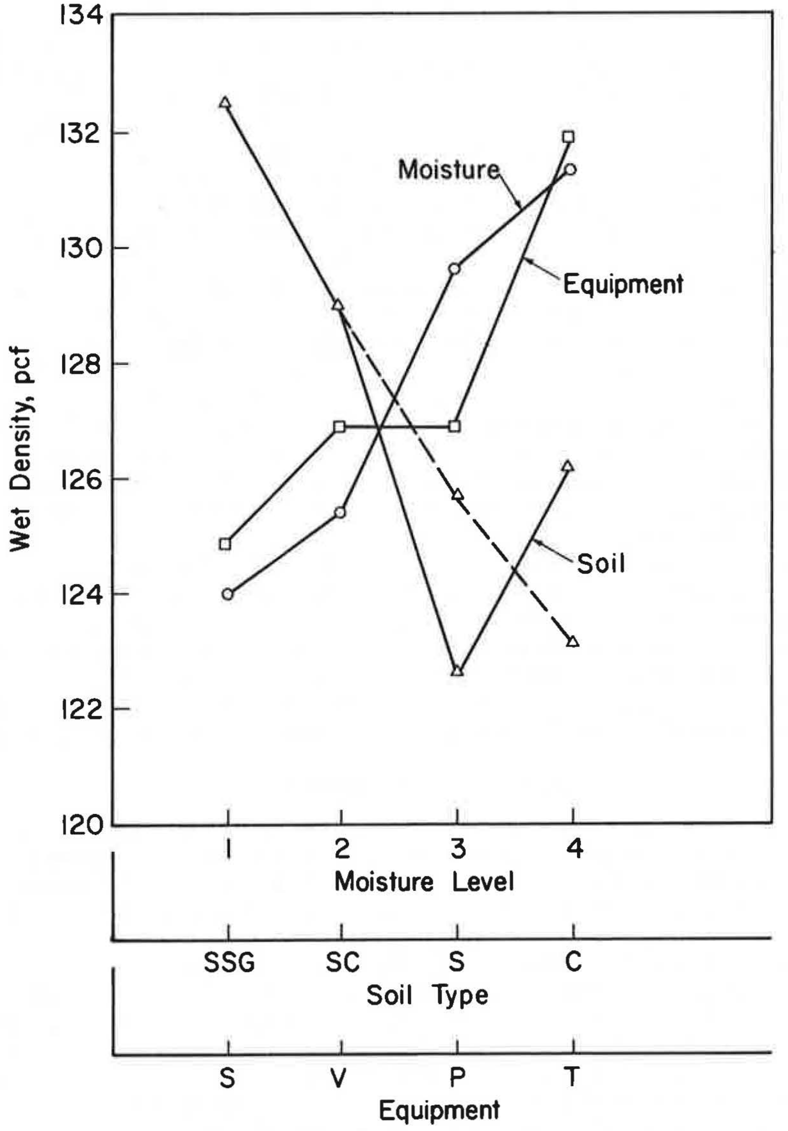


Figure 2. Final wet density variation with moisture, soil and equipment.

The joint effect ME was caused by a greater change in wet density over the existing moisture range for the pneumatic and vibratory rollers than for the sheepsfoot and segmented pad rollers. The MS effect was primarily a result of the difference in the relationship of the moisture range to the optimum moistures for each soil type (Fig. 1). The presence of an SE effect indicates that the relative performance of the rollers changes with soil types. The highest wet densities in each case were obtained with the segmented pad roller. For the silty clay, silt and clay the other three rollers gave lower values not significantly different from each other. However, for the silty sand and gravel the vibratory roller equaled the results with the segmented pad roller; the pneumatic roller did almost as well.

The Road Logger also showed a highly significant effect of T and C. The wet density decreased an average of about 5 pcf with an increase in T from 6 to 12 in., and increased an average of 3 pcf with an increase in compactive effort. The portable nuclear instrument showed the same trends, but the magnitude of the change was not large enough to be significant at the 10 percent level or better. The joint effect TE occurred because the overall reduction in wet density with T increase was caused primarily by the pneumatic and segmented pad rollers, whose effect was 6 and 8 pcf, respectively. There was a tendency for a decrease in wet density with C for the sheepsfoot roller, little effect of C for the vibratory roller and significant increases of 6 and 7 pcf for the segmented pad and pneumatic rollers, thus causing the CE effect.

The moisture density (w_d) with both nuclear instruments was affected by M and S at the highest level of significance. The trends were similar to those for the moisture content given in Figure 1. For the portable nuclear instrument, E was also a highly significant factor, but this was entirely caused by high readings obtained during the compaction of lifts with the sheepsfoot roller. The stripped measurements on these lifts were back in line with those for the other rollers—eliminating the E effect. The interaction effects for ME were present in each case, because of the interrelationship between the amount of compaction and the distribution of moisture content for each roller. The trends are the same for each roller, however. The MS effect reflects a difference in the distribution of moisture over the four levels with change in soil type. This is indicated to some extent in Figure 1. In several cases, the nuclear instruments indicated a change in ranking with respect to moisture density for the silt and silty clay as the moisture level changed.

The effect of C on moisture density was cause by a slightly higher (0.5 to 0.9 pcf) moisture density at the higher compactive effort than at the lower one. The significance of the joint effect CE resulted because the effect of C changed with the roller type. The pattern was the same as that for wet density, i.e., increase entirely due to the pneumatic and segmented pad rollers, no change for the vibratory roller, and a decrease for the sheepsfoot roller. Moisture density increases with both increased compaction and increased moisture content. Both probably contributed to the observed CE effect.

Penetrometer, Plate, CBR and Seismic Measurements

Moisture was the most significant factor influencing the strength and stiffness properties, since it was significant at the highest level for all such measurements, i.e., penetrometer, plate, CBR and seismic (Table 1). The effects of the remaining factors are not nearly as consistent; however, the factor ME was significant in all cases and the factors T, E, and MS were significant in three out of the four cases.

The general relationship between M and the final measurement of seismic velocity, penetration resistance, bearing plate load and field CBR is shown in Figure 3. All but the seismic velocity show a consistent decrease with increase in moisture level as expected. The seismic velocity increased up to the third moisture level and then decreased. The trend is more like that of dry density than strength or stiffness. The same trend was evident for the average of the growth measurements.

There was a consistent decrease in the measured properties for an increase in lift thickness from 6 to 12 in., except for the penetrometer. This trend for CBR held for all S, C and M. There was a TE effect because the decrease was all caused by the pneumatic and segmented pad rollers. There was no change for the vibratory roller and a small increase for the sheepsfoot roller. The magnitude of the decrease diminished with increasing M. This trend was not detected from the average of the growth measurements for seismic velocity, but it was exhibited by the final measurements for all M, S, C and E. Again the decrease diminished with M. The average of the bearing plate growth measurements showed a consistent decrease with thickness increase for all M, S, C and E, although the changes were not large enough to be statistically significant. The same general trend was exhibited by the final plate readings, except for a slight increase for the vibratory and sheepsfoot rollers.

The penetrometer did not show a significant T effect for either the growth or final measurements. However, there was an effect of T in relation to M and E. The resistance increased with T for all but the third moisture level. According to the TE effect, there was a large increase in resistance for the sheepsfoot roller and a consistent decrease for the segmented pad rollers. For the pneumatic and vibratory rollers, the

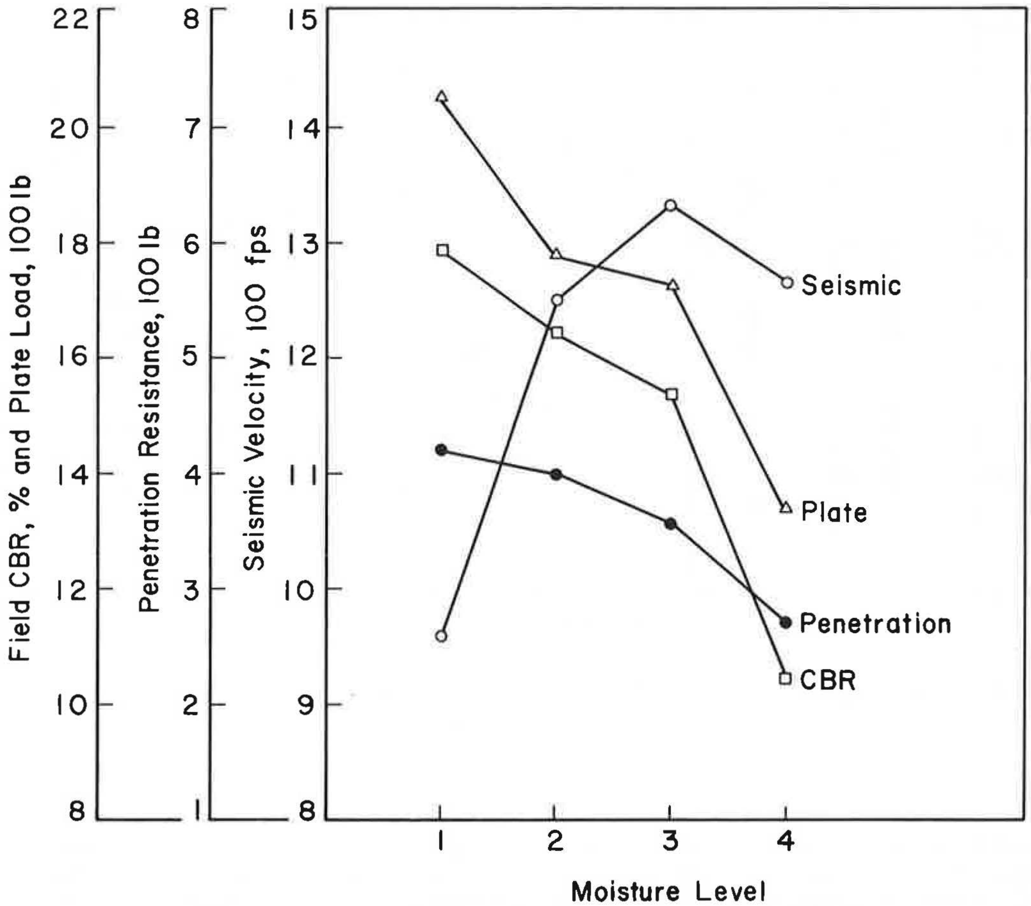


Figure 3. Penetration resistance, CBR, plate load and seismic velocity variation with moisture level.

change was either negligible or inconsistent. Considering the other three measurements, these observations suggest that side friction on the penetrometer shaft may have influenced the readings.

The factor C, itself, did not have a significant effect on any of the four measurements, although the general trend was an increase with increased effort; however, the joint effects with M, S and E were present in some cases. The significant effects of S and E are shown in Figure 4. The penetration resistance decreases with soil type in the order: silty sand and gravel, silty clay, silt, and clay, which is the same order as decreasing maximum dry density from the Proctor test. The seismic velocity follows the same trend for the first three soils, but the clay has the next to highest velocity. The effect of E on plate load, seismic velocity and penetration resistance is similar to that for wet density; i. e., the sheepsfoot, pneumatic and vibratory rollers gave results not significantly different from each other, while the values for the segmented pad roller were significantly higher.

The joint effect CE was significant for the bearing plate only. This was caused by a decrease for the segmented pad roller with increased compactive effort. This trend was also indicated by the penetrometer. The effect for the sheeps foot roller was mixed, while for the vibratory and pneumatic rollers there was a tendency for the properties to increase with compactive effort.

The final plate load increased with compactive effort for the two lowest moisture levels and decreased for the two highest levels, giving an MC effect. However, seismic velocity indicated an increase with compactive effort at all four moisture contents. The

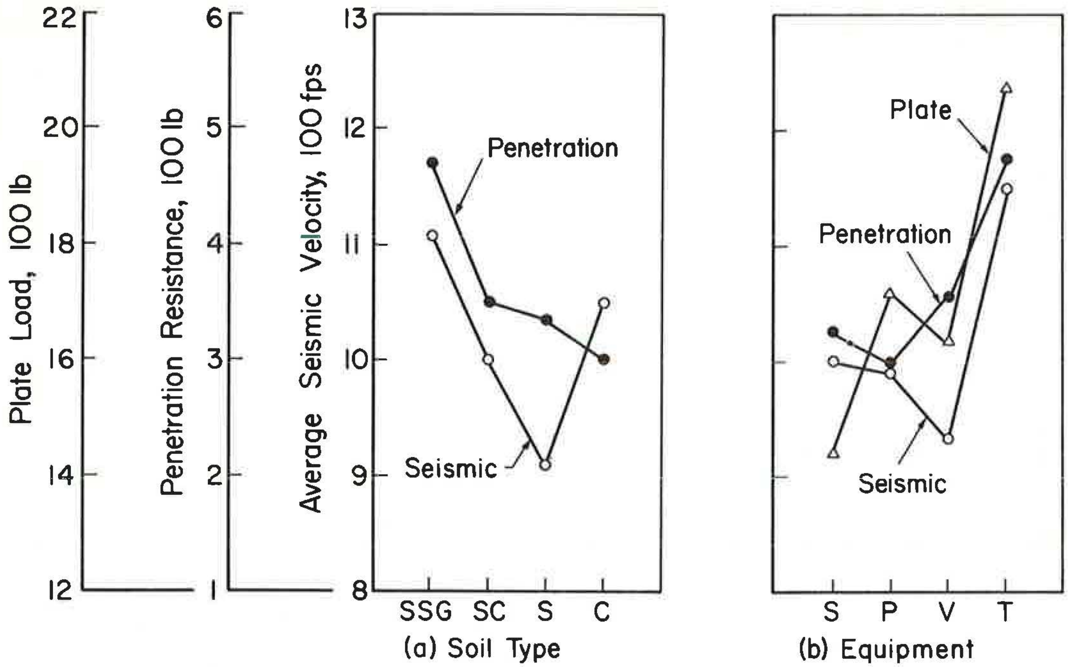


Figure 4. Penetration resistance, plate load and seismic velocity variation with soil type and compaction equipment.

plate load, penetration resistance and CBR decreased with M in the general manner shown in Figure 3 for the silty sand and gravel, silty clay and clay. However, these values did not change significantly for the silt, thus giving rise to an MS effect. The seismic velocity followed the general pattern of Figure 3 for all soils at the end of compaction, but there was little change for the average of the growth measurements for silt, thus creating the MS effect. For the plate load and penetration resistance, the trend in Figure 3 was followed for each roller, except that there was a maximum in the middle range of moisture levels for the vibratory roller giving an ME effect. The ME effect for seismic velocity occurred because of a change in the position of the maximum within the range of moisture.

TABLE 2

RANGE AND AVERAGE OF PROPERTIES FOR ALL EFFECTS

Measurement	Dimension	Range	Average	$\frac{\text{Range}}{\text{Average}}$ (%)
Moisture content	%	13.5	12.1	112
Field CBR	%	26.6	15.0	177
Wet density	pcf	29.8	127.6	23
Moisture density	pcf	10.9	12.6	87
Dry density	pcf	21.5	115.0	19
Penetration resistance	lb	529	364	145
Seismic velocity	fps	890	1192	75
Plate load	lb	1814	1719	105

Range of Values

The overall range and average of each of the measurements for the range of independent variables used in the tests are shown in Table 2. The average values were obtained by pooling all of the final measurements in each case for all of the test sections. The range was determined by subtracting the largest and smallest estimated mean values from the group representing all of the combinations of independent variables.

The largest change with respect to the average value occurs for the field CBR, whose range is 177 percent of its average. Next in decreasing order are the penetration resistance, moisture content and plate load which have ranges exceeding 100 percent of their average values. Moisture density and seismic velocity range about 80 percent of their averages. As expected, the property which changes the least with respect to its magnitude is density, the wet and dry densities having a range of about 20 percent of their average values. It might be expected that the best properties to measure are those whose percentage change is the largest; however, such a conclusion depends on the associated measurement variability. This latter question is examined in a companion paper (2).

CHARACTERISTICS OF GROWTH CURVES

The remaining independent variable, which has not been involved in the previous discussion, is the number of roller coverages. The mathematical model used in the analysis was so constructed that the shape of the growth curves for each measurement could be evaluated independently of the magnitude of the measurements. The factors influencing the shape of these curves are given in Table 3 in the same manner as was done for the measurements themselves in Table 1.

Portable Nuclear Measurements

The individual factors affecting the shape of the wet density growth curves obtained with the portable nuclear instrument were S, C and E. The curve shapes for S and E are shown in Figure 5. In order to establish the actual quantitative relationship between the curves in a set, the indicated mean value of each curve should be added. This will shift the curves vertically without a change in shape. By removing the mean values the comparison of shapes may easily be made; e.g., if the shapes are identical then the curves will be coincident regardless of the magnitude of the measurements.

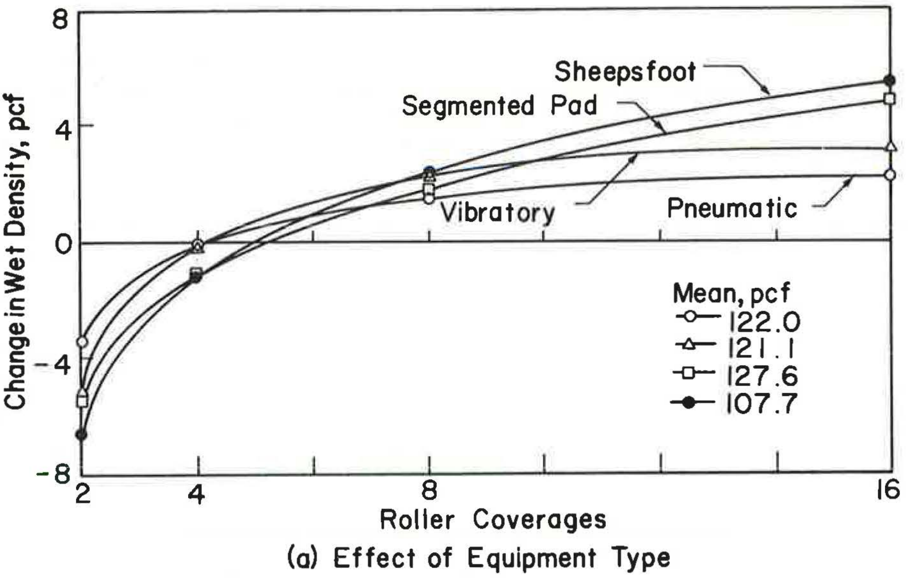
Figure 5a shows that the vibratory and pneumatic rollers had achieved their maximum amount of compaction by the end of 16 coverages for all four soils on the average, but density was still increasing at a significant rate for the segmented pad and sheepsfoot rollers. However, in terms of absolute density, the sheepsfoot roller was still at the lowest level at the end of 16 coverages, because its mean value of wet density was much lower than those for the other three rollers. Increase in density was still occurring

TABLE 3

RELATIVE SIGNIFICANCE OF INDEPENDENT VARIABLES ON GROWTH CURVE SHAPE

Measurement	Individual Effects					Joint Effects									
	M	T	S	C	E	MT	MS	MC	ME	TS	TC	TE	SC	SE	CE
Portable nuclear γ_w			2	6	1	6	4		5						
Portable nuclear w_d					6		6								
Portable nuclear γ_d				6	1		6		5					3	
Plate load	1				1								6	5	6
Pen. resistance	1		4				5	5							6
Seismic velocity	2					5	2		2						

Note: Error probability (%): 1 = 0.1, 2.= 0.5, 3 = 1.0, 4 = 2.5, 5 = 5.0, 5 = 5.0, 6 = 10.0.



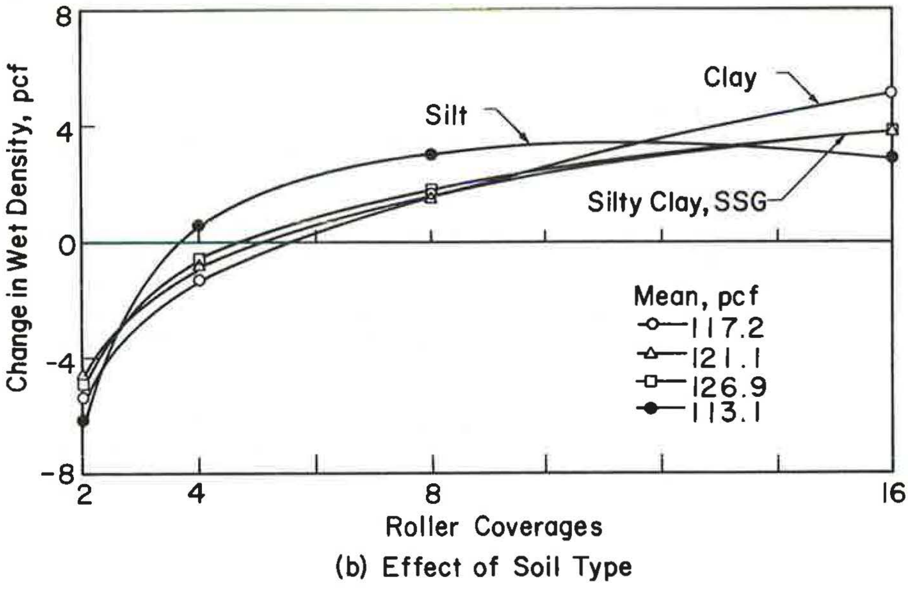
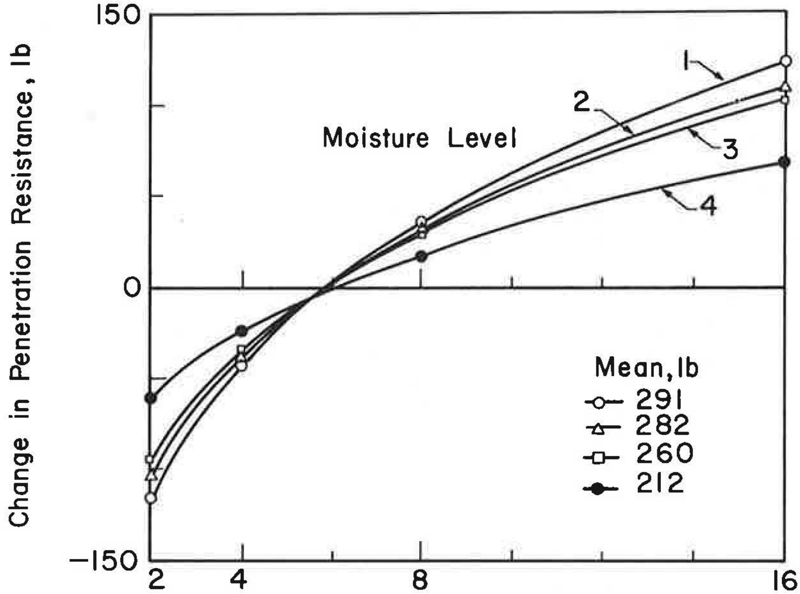


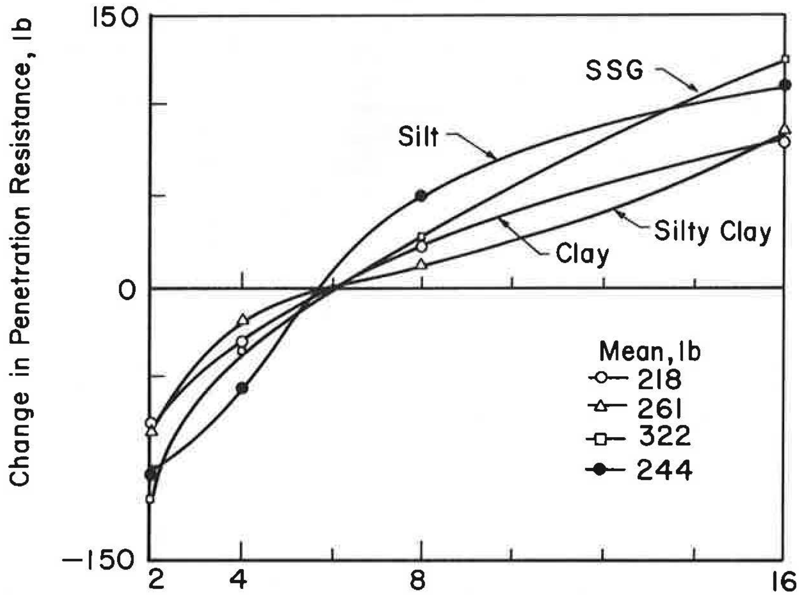
Figure 5. Average portable nuclear wet density growth curves for compaction equipment and soil type.

for the clay, silty clay and silty sand and gravel (Fig. 5b) at the end of 16 coverages, with the greatest change being in the clay. The silt compacted at a greater rate initially, but reached a maximum wet density at about 10 coverages and then began to show a decrease. The compaction occurred at a slightly greater rate for the higher compactive effort than for the lower, but the difference was small.

The MT effect indicates that with the 6-in. layer the rate of compaction increased with increase in moisture level; for the 12-in. layer, the reverse was true. Only the curve for the highest moisture in the 6-in. lift had reached a maximum by 16 coverages. The fact that M alone did not affect the shape suggests that the resulting



(a) Effect of Moisture



(b) Effect of Soil

Figure 6. Average penetration resistance growth curves for soil type and moisture level.

moisture-density curves had the same shape after every coverage on the average. For each individual soil, however, moisture did influence the shape. This MS effect may reflect the relationship of the actual moisture content to the optimum for the soil and compactive efforts involved. The ME effect indicates a change in the effect of moisture on the shape with change in compactor.

The growth curves for clay, silty clay, and silty sand and gravel were generally the same within each equipment group, although the clay density was always increasing at the greatest rate for each roller after 8 coverages. The silt curves were the most distinct and were the principal reason for the SE effect. The silt density was still increasing some after 16 coverages for the segmented pad roller. For the pneumatic and sheepsfoot rollers the curves leveled out after 8 coverages; however, for the vibratory roller there was a distinct decrease.

Theoretically, moisture density should increase with roller coverages in the same manner as wet density; however, the change is small enough with respect to the accuracy of the measurement that it can be considered constant in most cases. The change from 2 to 16 coverages for the pneumatic and segmented pad rollers was an increase of less than 0. 2 pcf and for the vibratory roller an increase of less than 0. 6 pcf. For the sheepsfoot roller, the moisture density decreased about 1 pcf. The MS effect was inconsistent, showing a decrease with coverage more often than an increase, the maximum change being 1 pcf in any case.

Penetrometer Measurements

The rate of increase of penetration resistance with coverage decreased continuously with moisture level increase (Fig. 6a). As a result, the total change in resistance between 2 and 16 coverages decreased with moisture increase. The shapes had much less curvature than those for density. The penetration resistance curves for the clay, silty clay, and silty sand and gravel were similar to those for wet density, except for less rapid initial rate of change (Fig. 6b). The shapes for silt were quite different, however. For this soil the penetration resistance changed in an approximately linear fashion up to 8 coverages and then continued to increase at a slower rate thereafter. At the higher compactive effort there was less difference between the shapes for each moisture level than at the lower effort.

The shapes of the penetration resistance curves for the four compactors were similar, although the segmented pad roller showed the greatest rate of increase at 16 coverages and the greatest change from 2 to 16 coverages. The lowest rate and smallest change occurred for the pneumatic roller. The CE effect was produced by the sheepsfoot roller, whose growth curve changed in shape with change in compactive effort. The overall trend was the same as that for the other rollers, but it had a double curvature which reversed direction with change in C.

Bearing Plate Measurements

The most significant factors influencing the bearing plate growth curve shape were E and M. As with the penetrometer, the rate of increase of plate load decreased with increase in moisture (Fig. 7a). The biggest difference occurred between the third and fourth moisture levels. The pneumatic and segmented pad rollers gave the greatest increase and rate of increase of the four compactors over the range of 2 to 16 coverages. Next in order was the vibratory roller and then the sheepsfoot roller (Fig. 7b). In most cases the plate load was still increasing at a significant rate after 16 coverages.

The difference in the growth curve shapes for the four soils decreased with increased compactive effort. The same was true for the compaction equipment. The difference in growth curves for the sheepsfoot roller in the four soils was small. The vibratory roller showed the greatest change from 2 to 16 coverages for the clay and essentially identical results for the other three soils. The pneumatic roller showed the greatest change for the silt and the same results for the other three soils. All four soil curves were different for the segmented pad roller.

Seismic Velocity

The only individual factor affecting the shape of the seismic velocity growth curves was moisture. The greatest change in velocity occurred for the two intermediate mois-

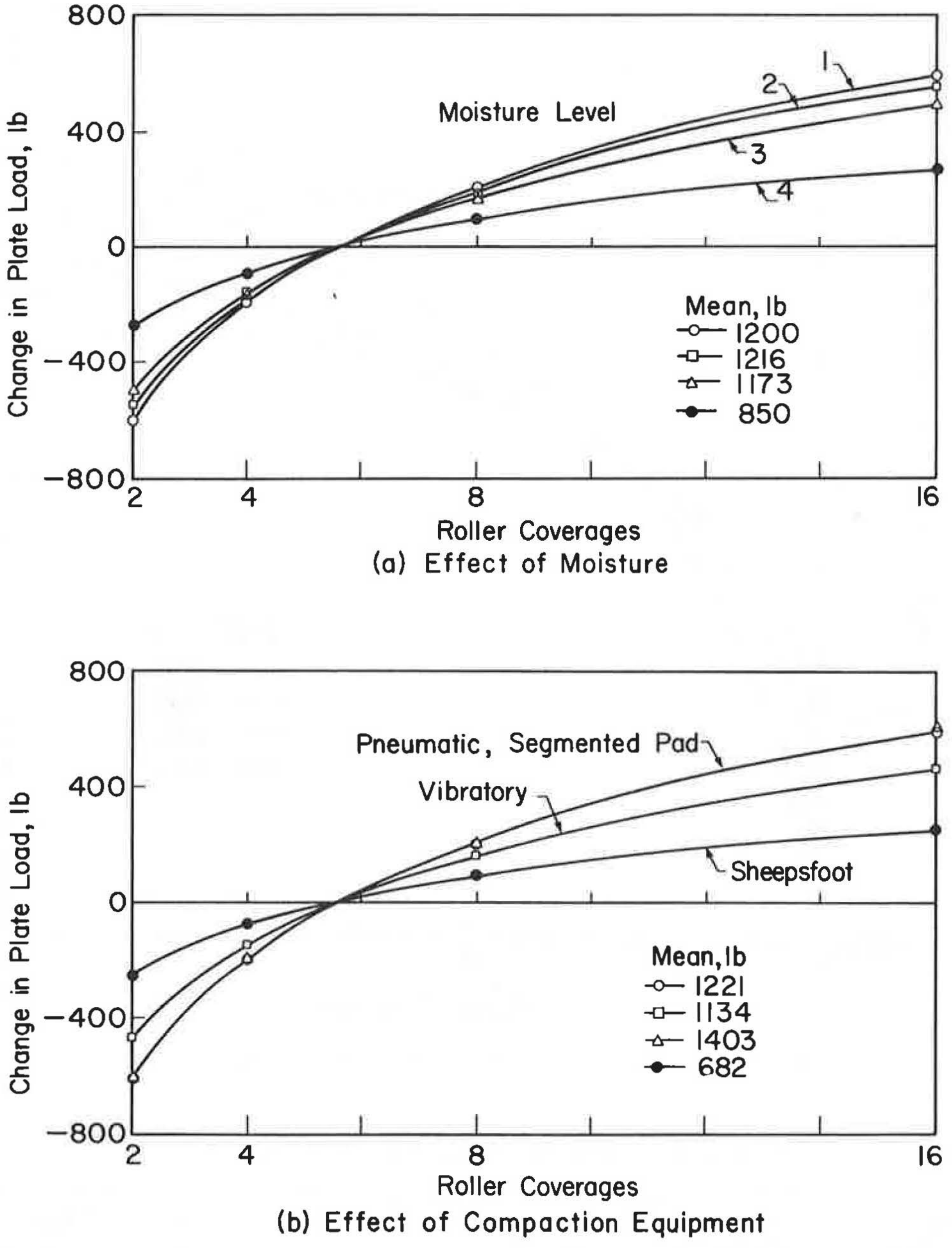


Figure 7. Average bearing plate growth curves for moisture level and compaction equipment.

ture levels and the highest moisture level had the smallest change. The trend was an increase in the magnitude of the change from moisture level 1 to 3 and then a decrease for moisture level 4 (Fig. 8). These differences were accentuated for the 6-in. lift thickness and decreased for the 12-in. thickness, giving an MT joint effect. The same trends generally held for all soils except the silt in which case the trend was inverted; i. e., the magnitude of the change decreased from moisture level 1 to 3 and then increased again for moisture level 4. In all cases the seismic velocity was still increasing after 16 coverages, except for the sheepsfoot roller at the highest moisture content. In this case the velocity continued to decrease beginning with coverage 2.

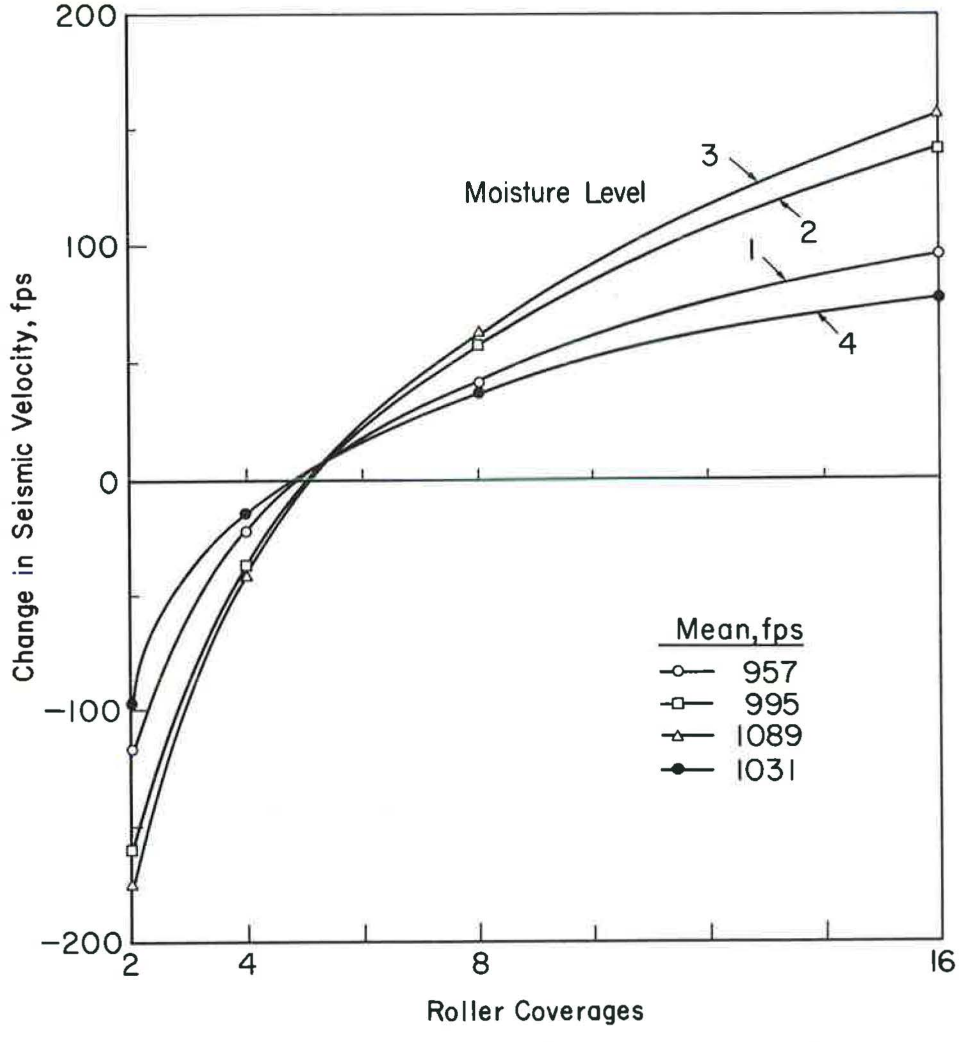


Figure 8. Average seismic velocity growth curves for moisture level.

MOISTURE-DENSITY-STRENGTH RELATIONSHIPS

Standard (T-99) and modified (T-180) Proctor compaction tests were performed in the laboratory on samples of soil taken from the stockpiles used for the field studies. In addition, a T-180 test was performed on a sample from each test section taken after mixing and just prior to the first coverage of the roller. These samples were compacted at the same moisture content as the test section and involved the identical preparation procedures. The average results from the field Proctor tests are compared in Figure 9 with the peak points from the laboratory tests.

The four subgrade soils have distinctly different moisture-density-strength characteristics. The field Proctor dry densities (T-180) lie midway between the T-99 and T-180 values obtained in the laboratory for the clay, silty clay and silty sand and gravel. The field values for the silt correspond approximately to the T-99 values from the laboratory tests. The optimum moisture contents appear to occur at about the same percent saturation as in the laboratory tests. The CBR values from the field T-180 tests correspondingly lay in a range midway between those for the laboratory T-99 and T-180 tests (not shown in Fig. 9), as would be expected considering the relationship of

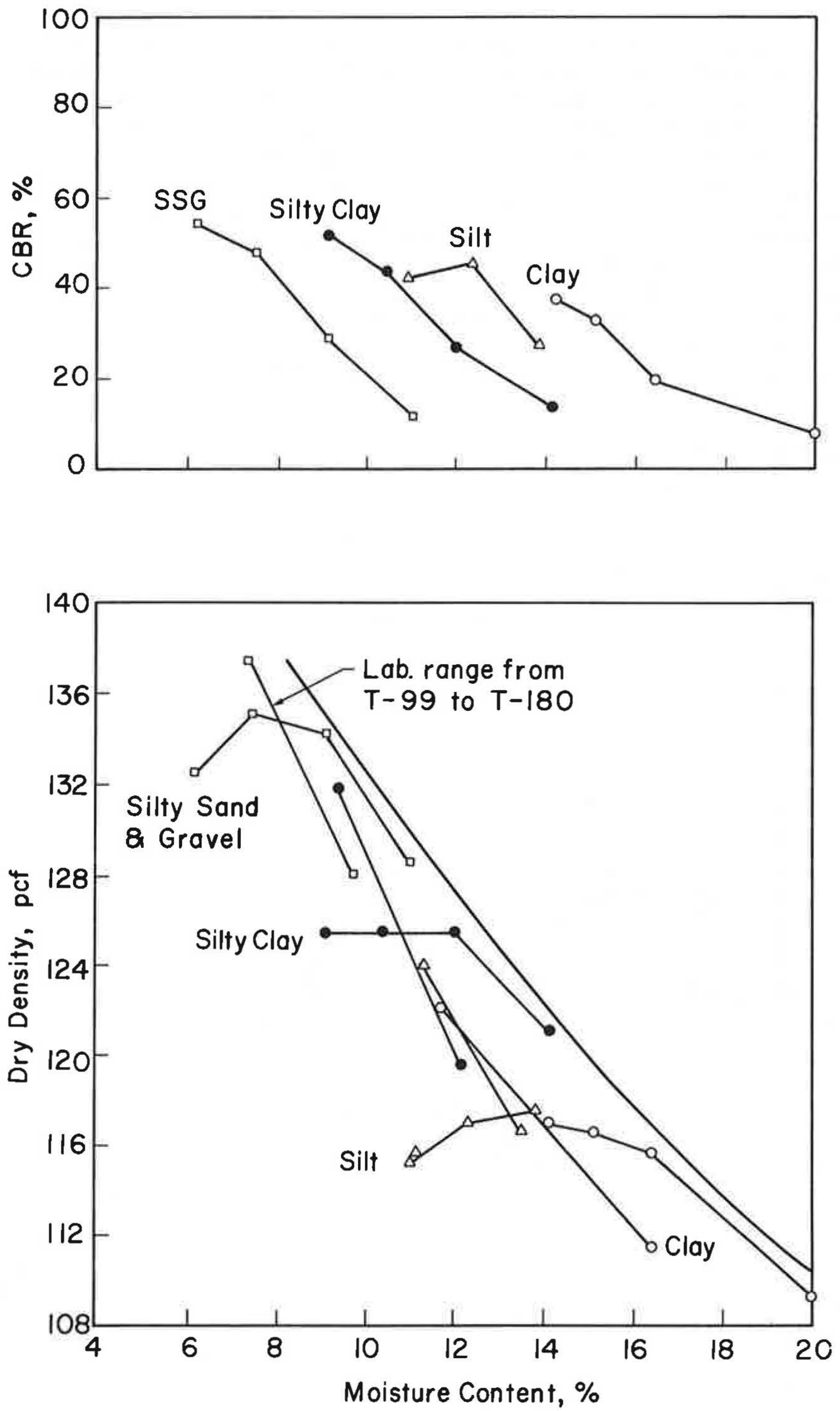


Figure 9. Moisture-density-CBR relationships from field modified Proctor tests.

the dry densities. However, the field CBR curves appear to be shifted toward higher moisture contents relative to the laboratory values than might be expected, based on magnitude of the related density alone.

An examination of the individual data from which the average values in Figure 9 were obtained showed that the discrepancy between the field and laboratory results could be explained to a large extent by the averaging process used. The analysis assigns all values of moisture content and dry density into four groups, one for each level of moisture in a lift. However, the actual values of moisture in each level varied enough between lifts to overlap those in other levels. Therefore, when each level is averaged, because of the concave downward shape of the moisture-density curves the resulting curve will be on the low side of the range of data. The analysis of variance model used needs further study in an attempt to find a means of overcoming this limitation. It is the field data in the form shown in Figure 9 which should be used for comparison with the results on the lifts, because both sets of data were analyzed using the same method.

The average moisture-density curves from the compacted lifts for each of the four soils are compared with the corresponding field T-180 curves in Figure 10. The lift

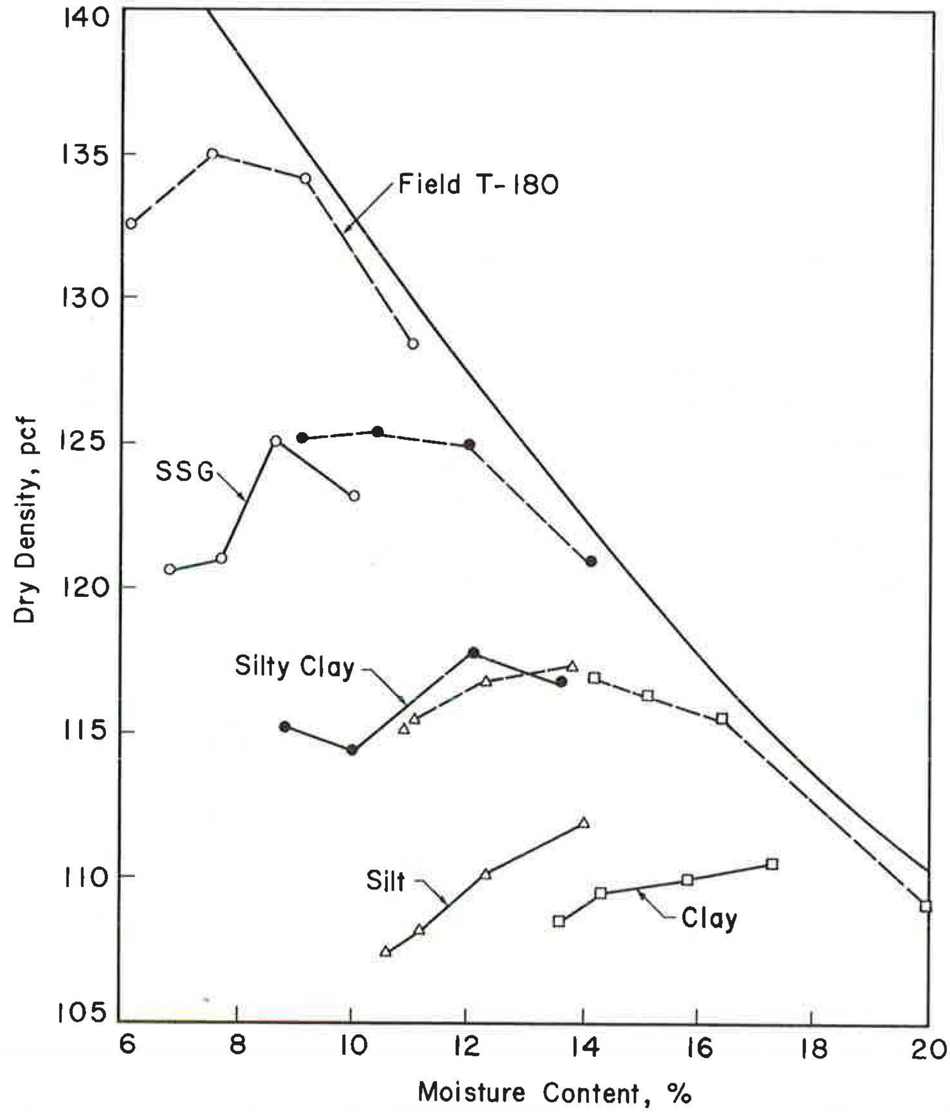


Figure 10. Average moisture-density relationships for each soil from compacted lifts compared with field Proctor tests.

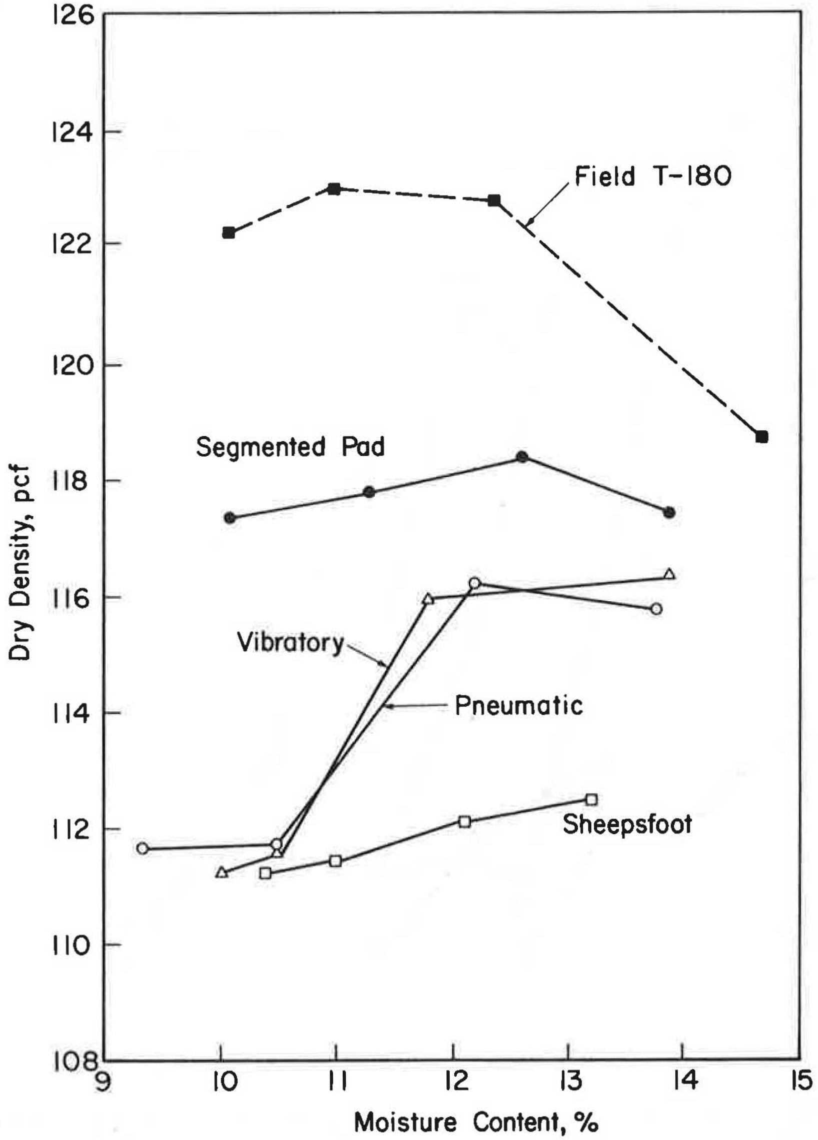


Figure 11. Average moisture-density relationships for each compactor.

curves appear to lie primarily on the dry side of optimum for the average conditions involved, and the average dry densities are substantially below those from the T-180 tests.

The moisture-density curves for each compactor averaged for all other conditions are shown in Figure 11. These curves confirm the dry side compaction for all rollers. The biggest change in dry density with moisture level occurred with the pneumatic and vibratory rollers.

The field CBR values for each soil as a function of moisture content are compared with the corresponding values from the field Proctor tests in Figure 12. The Proctor values are much higher than the values from the compacted lifts at the low end of the moisture range, and converge to similar values at the wet end of the range. This is the same manner as the CBR curves would be related for two different compactive

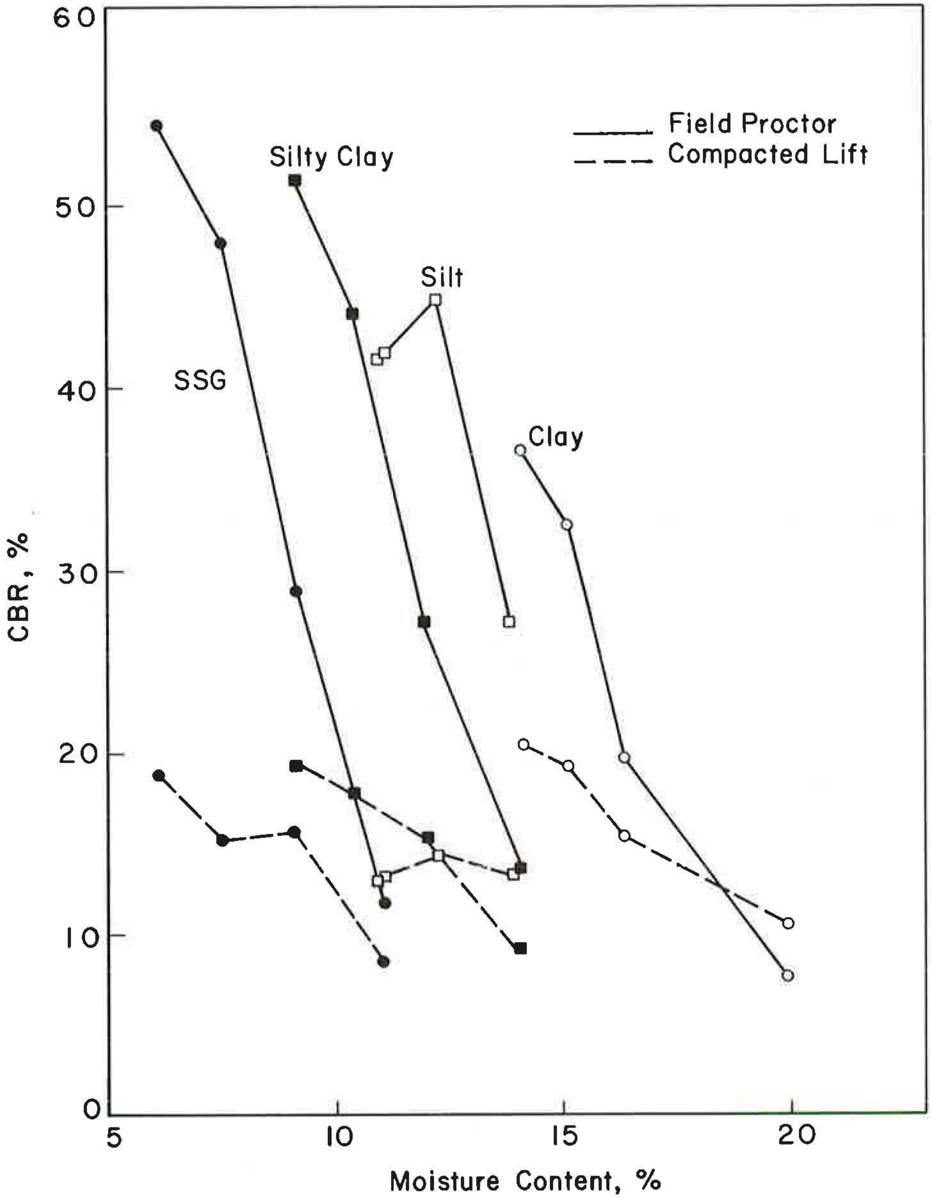


Figure 12. Comparison of field CBR on lifts and CBR from field Proctor tests for each soil.

efforts in the same soil. The magnitude of the difference is approximately that which would be expected on the basis of the difference between the corresponding densities (Fig. 10). The resemblence in shape between the pairs of curves is evidence of similarity in the compaction effects of the rollers and the Proctor hammer.

Previous discussion has dealt with average conditions. Comparisons can also be made for any combinations of the independent variables by superimposing the effects of each upon the average. In Table 4, the maximum dry densities based on the nuclear measurements are listed for each soil and compactor combination. They were obtained by adding to the average values for each SE combination after 16 roller coverages the increases caused by the most favorable compactive effort, lift thickness and moisture

TABLE 4

MAXIMUM COMPACTION FOR EQUIPMENT-SOIL COMBINATIONS

Compaction Equipment	Soil	T-180	T-99 (Estimated)		Maximum Dry Density (pcf)	Percent T-180	Percent T-99 (Estimated)
Pneumatic	Clay	117	107		118	101	110
	Silty clay	126	114		122	97	107
	SSG	135	126		127	94	101
	Silt	118	110		115	97	105
				Average	120	97	106
Vibratory	Clay	117	107		111	95	103
	Silty clay	126	114		119	95	104
	SSG	135	126		128	95	101
	Silt	118	110		112	95	102
				Average	117	95	103
Segmented pad	Clay	117	107		119	102	111
	Silty clay	126	114		125	99	110
	SSG	135	126		130	96	103
	Silt	118	110		119	100	108
				Average	123	99	108
Sheepsfoot	Clay	117	107		109	93	102
	Silty clay	126	114		116	92	102
	SSG	135	126		120	89	95
	Silt	118	110		111	94	101
				Average	114	92	100

level in each case. The maximum dry densities from the field T-180 tests were obtained from Figure 9. An estimate of the corresponding T-99 values was made by subtracting the differences observed in the laboratory Proctor tests. These values were used to compute the maximum dry density in percent of T-99 and T-180 for the compacted lifts.

The percent of T-180 obtained ranged from 89 to 102, the compactor order with respect to increasing values being sheepsfoot, vibratory, pneumatic and segmented pad. This ranking was essentially the same for all four soils, although the magnitude of the differences between rollers changed. The estimated percent of T-99 ranged from 95 to 111 with the compactor ranking remaining the same as for T-180. The same computations for the average dry densities for each roller give as percent T-180 the values 90, 92, 92 and 95 for the sheepsfoot, vibratory, pneumatic and segmented pad rollers, respectively. For percent T-99 the corresponding values are 98, 100, 100 and 103. Therefore, the average compaction is equivalent to T-99 and the ranking is the same as that for maximum dry density, except that under average conditions the pneumatic and vibratory rollers provide the same results.

SUMMARY AND CONCLUSIONS

This paper dealt with the properties of compacted soils based on observations from field tests in which the soil type, moisture content, lift thickness, compaction equipment and compactive effort were the main parameters varied. In view of the nature of the field test program, it is believed that the conclusions will have direct application to construction practice. Only the major effects could be detected from the resulting data because of the large variability encountered in the tests, principally as a result of moisture control difficulty. The behavior which could be observed will thus be pertinent to construction operations and those details which could not be distinguished because of the variability are probably not of practical significance. The test plan was based on a statistical model which permitted separation of real effects from random variability. An examination of appropriate field measurements verified the accomplishment of this objective.

The range of moisture contents selected bracketed the T-99 (standard Proctor) and T-180 (modified Proctor) values for each soil. The results showed that the average test conditions produced a level of compaction equivalent to the T-99 effort; thus, the measured properties represent behavior more on the dry side of optimum than on the wet side. Further study of the data should lead to a better understanding of effective compactive effort for a wide variety of compactor types. This is a subject which is not adequately understood at present, especially for vibratory rollers, and limits the ability to predict the relative performance of different field compaction equipment.

The observed properties of the field compacted soils appeared similar to those exhibited by laboratory compacted specimens. For the range of conditions involved, moisture was by far the most significant factor influencing the measurements. Next was the soil type and then in descending order of importance were compaction equipment, lift thickness and compactive effort. However, the relative importance of each of these parameters depended a lot on the specific combinations considered. There were no significant interactions between soil, lift thickness and compactive effort, and thus the effect of any one of these three parameters did not change with change in the others. Moisture and thickness had little interaction as well, leaving compaction equipment as the only parameter whose effect changed with a change in lift thickness. Another important observation was that the relative effectiveness of each roller did not change appreciably with change in soil type.

Increasing lift thickness from 6 to 12 in. caused a decrease in density of up to 6 to 8 pcf for the pneumatic and segmented pad rollers. No significant effect was observed for the vibratory and sheepsfoot rollers. The same trends were observed for the bearing plate, seismic and CBR measurements. The increase in compactive effort for the pneumatic and segmented pad rollers caused the largest increase in the measured properties. The change with the vibratory and sheepsfoot rollers had little effect. It will be recalled that the single-lift test section tended to be an unfavorable factor for the sheepsfoot roller.

Superimposed upon all of the foregoing effects, which are based on a constant number of roller coverages, is the effect produced by a change in the number of coverages. As a general rule, the magnitude of the measured properties increased with coverages, but in decreasing amounts. Only for wet density was there evidence of a leveling off of the growth curves in less than the 16 coverages considered. However, in the majority of cases wet density was increasing at a significant rate after four coverages and continued to increase over the entire range with as much as 3 to 5 pcf change from 8 to 16 coverages. When maximums were reached they occurred after 8 coverages.

The plate load and penetration resistance measurements generally showed no tendency to level off within 2 to 16 coverages, and half of the total increase in this range usually occurred after 6 coverages. The seismic velocity growth curves were intermediate between these and the wet density curves, but in all except a few cases exhibited no leveling off up to 16 coverages.

As comprehensive as this study has been there are still a number of important factors which have not been considered. Among them are foundation conditions, multiple layer compaction, long-term environmental influences, and roller speed. In addition, considerable analysis can still profitably be done with the data already collected. The statistical model should be further studied to provide a means of incorporating moisture as a continuous variable, and to include physically meaningful coefficients representing effective compactive effort and relationships between moisture and the measured properties. Field test programs should be a continuing effort because they provide direct application to practical field problems as well as an opportunity to apply fundamental knowledge gained through basic research and, as a result, serve to bridge the gap between theory and practice.

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Current Specifications, Field Practices and Problems in Compaction for Highway Purposes

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This paper attempts to summarize the current status of highway specifications and field practices for compaction of embankments, subgrades and granular bases. The information has been obtained from the published standard specifications of the 50 states, and from an extensive interview program with state highway engineers. Construction specifications and procedures for embankments, subgrades and granular bases are summarized and followed by discussions of the problems related to the practical application of the specifications of field construction. Quality control procedures and related problems also are discussed.

•FROM July 1964 to August 1966, North Carolina State University, under the sponsorship of the U. S. Bureau of Public Roads, conducted a comprehensive review (1) of the current state of knowledge in regard to the compaction of soil and rock materials for highway purposes. As one part of this study, a compilation and evaluation of current state highway specifications and field practices for earthwork construction was undertaken. This paper presents a summary of the findings.

Primary information regarding specifications was obtained from the most recent editions of each state's standard highway specifications and special provisions for earthwork construction. To supplement this published information, a comprehensive program of personal interviews was conducted. State highway department offices were visited in 22 states, selected to provide a reasonable cross section of geographic, climatic and soil conditions. In each state, two to six individuals were interviewed, including materials engineers, construction engineers, field and laboratory soils engineers and geologists. Interviewees were questioned regarding local compaction problems and techniques for overcoming them, interpretation of specifications in practice, compaction control procedures and problems, and suggestions for improvements of specifications and practices. When time permitted, visits to construction sites or problem areas were arranged. The interviews provided considerable insight into the practical aspects of earthwork construction and the problems that are of major concern to highway engineers.

Several limitations of the interview program should be recognized. No interviews were conducted in approximately half of the states. In the states that were visited, the interview program was limited to engineers in the central office because of limitations in both time and funds. The interviewees frequently commented on the variation in practices and attitudes from one district to another within their state; thus, the opinions expressed in the central office may not be representative of the attitudes at the district level. This was generally attributed to the autonomy of the district engineers. In addition, it was apparent that some individuals discussed their problems, practices, and the enforcement of their specifications more frankly than others. Nevertheless, certain general impressions of universal problems and practices became apparent through the interview program.

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

The discussion of current specifications and practices is separated into two main sections. First, field procedures and compaction requirements are presented. For each pavement component, a summary is given of the information obtained from the published specifications. The summary is followed by a discussion of the findings of the field interview program pertaining to the particular aspect of the specifications. The second major section discusses current quality control procedures. The paper concludes with a summary of major field problems.

CURRENT SPECIFICATIONS AND PRACTICES

Compaction specifications may indicate the procedure by which the compaction is to be accomplished, the required quality of the compacted materials, or some combination of procedure and required results. The specified procedure may include moisture control, lift thickness, type and size of compaction equipment, and the number of coverages of the equipment. The quality of the compacted material generally is specified in terms of dry density, which usually is expressed as a percentage of the maximum dry density achieved in a specified laboratory compaction test.

Specifications commonly are referred to as "procedural" or "end-result" specifications, depending on whether or not density requirements are specified. However, these terms may be somewhat misleading. End-result specifications usually include some procedural requirements. Lift thickness and moisture control commonly are included, and equipment type and size are sometimes indicated. However, the equipment requirements may be quite general, and the number of coverages, or required compactive effort, is omitted. On the other hand, the procedural specifications will include the number of coverages or a relatively simple visual acceptance criterion, such as the walk-out of a sheepsfoot roller.

The addition of a minimum-density requirement to a detailed procedural specification generally is considered undesirable because of the potential contractual problems. Legal problems may result if the contractor adheres closely to a detailed procedure and yet is unable to achieve the required density. However, several states are successfully combining a minimum procedural requirement with a density requirement. The concept of minimum compactive effort is introduced to insure uniformity of compaction and to reduce the number of density tests required.

A comparison of current specifications with those compiled in 1960 indicates a general trend toward greater reliance on the end-result or density requirement. Current usage for each component of the road section will be discussed in subsequent sections on embankments, subgrades, and granular bases.

Equipment

Approximately three-fourths of the states include some minimum equipment standards in their specifications. Frequently standards may be given for only one type of compaction equipment, usually smooth-wheeled or pneumatic-tired rollers, or for construction of one component of the pavement section, most commonly the base course. In addition, most state specifications include a provision that equipment must be satisfactory to or approved by the engineer. In practice, these minimum equipment standards appear to be of little practical concern to highway engineers. Most contractors are using adequate equipment with regard to both size and type suitable for each soil type encountered. Consequently, inspectors rarely are called upon to exercise their authority regarding approval of compaction equipment.

In practice, a wide variety of types of field compaction equipment is being used including smooth-wheeled, pneumatic-tired and sheepsfoot rollers, vibratory compactors, and specialized equipment that utilize combinations of compactive actions, such as the vibratory sheepsfoot roller. For cohesive soils, sheepsfoot and pneumatic-tired rollers still are most commonly used. However, for granular soils, there is an increasing utilization of vibratory compactors. This equipment apparently is providing efficient and satisfactory compaction of such materials with a minimum of problems. Advantages attributed to vibratory compaction of granular materials include the effective compaction of thicker lifts than is possible with conventional rollers and the reduction of degradation effects in crushed-limestone base course materials. However, the magnitudes of these effects remain subject to debate.

Although many states require that equipment specifically designed for compaction be at each compaction job site, in practice a considerable amount of compaction still is accomplished by hauling equipment. It is recognized that hauling operations can produce significant densification of earth fills. However, compaction solely by hauling operations is considered undesirable because uniform coverage and, as a consequence, uniform density generally are not achieved. To overcome this difficulty, some states permit compaction by hauling equipment together with supplementary rolling by compaction equipment to improve the uniformity. It should be noted that the compaction equipment must produce higher stresses in the fill than the hauling equipment if greater uniformity is to be obtained.

The heavy loads imposed by hauling equipment create a major problem in some embankment construction. In many states examples were cited of heavy hauling or paving equipment causing stability failures in compacted embankment and subgrade materials that had already satisfied compaction specifications. Almost all of the cited problems occurred in silty materials that are extremely sensitive to moisture and density conditions. The wheel loads from this equipment may produce higher stresses in the compacted soils than the stresses to be anticipated from traffic loads after the road is in service. It can be anticipated that these problem with heavy equipment will become more common in the future unless corrective measures are employed.

Embankments and Subgrades

The current trend for embankment and subgrade specifications is to minimize the procedural requirements and to place greater reliance on density requirements. For subgrades, only 4 states do not have density requirements. All four, Maryland and 3 New England states, merely specify compaction with a 10-ton roller. Several other states specify minimum equipment together with density requirements. Three other states, for some types of work, specify minimum equipment without density requirements. However, the vast majority of states rely almost entirely on density requirements for subgrades.

For embankments, all states have density control specifications. However, approximately 25 percent of the states have alternate specifications for compaction without density control that are used for certain types of construction. In these cases, the specified procedure may be the minimum number of passes of a specific piece of equipment or the use of suitable equipment for compaction to the visual satisfaction of the inspector.

In practice this kind of specification generally means using a sheepsfoot roller until it walks out or a pneumatic roller until there is no further observable densification of the soil. In some instances compaction by hauling equipment, usually followed by proof rolling, is permitted.

For embankment construction, the maximum lift thickness is specified, usually expressed in terms of the loose or uncompacted material. Almost 60 percent of the states specify the maximum uncompacted thickness as 8 in. and an additional 15 percent specify 6 or 9 in. Some allow 12-in. lifts in all materials, while others increase the allowable thickness to 12 in. for granular soils or rocks. Occasionally lift thicknesses greater than 12 in. are permitted when rock is encountered. In regions of high rock content, lift thicknesses may be increased to permit burial of large boulders near the bottom of embankments. Four states specify lift thickness requirements in terms of the compacted thickness, specifying either 6 or 8 in. as the compacted thickness.

All but 10 states specify the minimum depth of subgrade compaction. More than 60 percent of the states specify compaction to a minimum depth of 6 in. The remaining states specify depths of compaction varying from 4 to 12 in. A few states require subgrade compaction to a depth of 18 in. or greater when rock is encountered.

Lift thickness was not an area of major concern to most of the highway engineers who were interviewed.

Density Requirements for Embankments and Subgrades—The density requirements for embankments and subgrades are based predominantly on the AASHO T-99 Compaction Test or a similar test with an approximately equivalent compactive effort. For both subgrades and embankments, the most common requirement, used by almost half of the states, is 95 percent of the maximum dry density obtained in the T-99 test. For subgrades approximately 10 states use 100 percent of T-99, and only 2 use 90 percent. On the other hand, for embankments only 2 states require 100 percent and 11 require 90 percent of T-99. In all instances, the subgrade density requirements are equal to or greater than those required in embankments. In approximately 20 percent of the states the embankment requirements in the upper 1 to 6 ft of the embankment, depending on the state, are equivalent to those of the subgrade, and are less for the remaining depth of the embankment. Less than 5 states use the AASHO T-180 test for embankments and subgrades. In addition, California uses a special impact test that uses a compactive effort intermediate between the AASHO T-99 and T-180 efforts, which produces densities approximating those from the T-180 test. Several states base density requirements on a relative density concept in which the required density is specified in relationship to a maximum and a minimum density for the material. One example of this technique is the Texas compaction ratio method.

Approximately 10 states vary density requirements with soil type, magnitude of maximum dry density, and height of fill. When the requirements vary with maximum dry density, the percent of maximum density required decreases as the magnitude of the maximum dry density increases. Because the maximum dry densities usually are higher for granular soils than for cohesive soils, the required percent of maximum density usually is lower in granular soils than in cohesive soils. One of the states that varies density requirements with soil type also reduces the density requirements for granular soils. However, the other states that vary density requirements with soil type increase the percent of dry density required for granular soils. Inconsistencies among adjacent states sometimes develop. For example, both Illinois and Indiana generally require 95 percent of AASHO T-99; however, for granular soils Illinois increases its requirements to 100 percent and Indiana reduces its requirements to 90 percent. In almost every case, the variations in density requirements have resulted from judgment and experiences with local construction practices rather than from theoretical considerations. For instance, in Ohio and Indiana, the higher density requirements for cohesive soils were attributed to experiences that indicated more stability problems were encountered with cohesive soils. In Nebraska, higher density levels are being used with granular materials because they can be easily attained, whereas the same density levels cannot be achieved in cohesive soils without extremely close moisture control and much additional cost. In addition, it was felt that the cohesive soils would not maintain the high density in service. In Colorado, lower density levels are used for expansive clays.

In the interviews, most highway engineers appeared satisfied with their current density requirements and made almost no mention of a need for higher density. In fact, density requirements in Georgia have actually been reduced recently and to date satisfactory results are reported. In other states where silty soils are prevalent the feeling was expressed that higher density, and in some instances current density levels, will not be maintained in service even if they can be achieved during compaction. Experiences were cited, for example, in the loesses of Iowa and Nebraska, for which the compacted density was reported to have been reduced after exposure to environmental conditions and traffic. However, the reports generally involve observations of stability loss rather than density loss. No evidence was reported to indicate clearly whether the stability loss was caused by a loss of density or a loss of strength caused by increasing degree of saturation at constant or even increasing density. The latter explanation is strengthened by reports in the same states of instability of silty embankments immediately after moisture-density requirements have been satisfied.

As noted earlier several states require higher densities, which are usually equal to their subgrade requirements, in the upper 1 to 6 ft of an embankment. This apparently is done because the stresses produced by wheel loads are greater in the upper regions of the embankment. In some states the feeling was expressed that the density achieved in the upper parts of the embankment can be economically produced uniformly throughout the entire embankment. Because the contractor will furnish equipment that can provide the higher density levels with a reasonable number of coverages, it is reasoned that the entire embankment can easily be compacted to the same density level. Consequently, except for very high fills there appears to be very little concern for variations in density requirements as a function of position within an embankment.

In a few states, the embankment density requirements are increased throughout the entire embankment when the embankment height exceeds some predetermined elevation or when the embankment is subject to flooding. In the first case, the increased densities are used to offset the higher overburden pressures. In the second case, the higher densities are required to offset the loss in strength that will accompany saturation caused by flooding.

Although density control procedures will be discussed more fully in later sections, it should be noted that some of the lack of concern for more exact density requirements is related to the reliability of the percent compaction determinations. These computations can be no more accurate than the field density tests and the laboratory compaction test on which they are based. Problems related to testing procedures are discussed in the second part of this paper.

Moisture Control for Embankments and Subgrades—A statement regarding moisture requirements is included in the specifications for embankments in all but 2 states and for subgrade in all but 9. However, in approximately 60 percent of the states the moisture conditions for both embankment and subgrades are specified in a qualitative manner which leaves the interpretation largely to the judgment of the inspector. Qualitative statements include "to the satisfaction of the engineer," "as required by the engineer," and "as required for compaction." Quantitative statements for moisture limits relative to the optimum moisture content for the soil are specified by approximately 40 percent of the states for embankments and approximately 25 percent for subgrades. Some of these quantitative requirements merely specify "at optimum moisture content" or "as near as possible to optimum moisture content." In practice, these statements become qualitative by interpretation of the inspector. However, the majority of the states using quantitative moisture requirements specify minimum and/or maximum moisture conditions. Either the maximum or the minimum limit may be omitted in a number of states because of the predominant climatic conditions. For example, states in arid regions frequently specify lower moisture limits but not upper limits. In wet regions, the converse may be true. Occasionally moisture control requirements are waived for granular soils and rock.

The importance of maintaining proper moisture contents during compaction apparently is recognized by almost all practicing highway engineers. However, there are many differences of opinion regarding the practical procedures for, and even the feasibility of, controlling moisture. Specifications that state moisture control should be "to the satisfaction of the engineer" or "as required by the engineer" are difficult to enforce. They rely entirely on the engineer's judgment which may be questioned by the contractor. Wide variations in interpretation may exist among inspectors within a state. Indirectly construction costs may be increased because the contractor may increase his bid to allow for the uncertainties involved with this type of control. On the other hand, specifications that indicate specific moisture limits with respect to the optimum moisture content from a standard laboratory test create other problems. Inspection becomes more costly and more time consuming, and delays to the contractor may result. Because the compaction characteristics differ for various soil types, it is difficult if not impossible to specify one moisture content range that will be satisfactory to all soil types.

General practice regarding moisture control varies with state, soil type, and climatic conditions. As would be expected there is little concern for moisture control in granular soils. For cohesive soils the closest control is exercised for soil types, such as silts and swelling clays, which through experience have caused the most difficulty. The moisture-density curves for silts have sharp peaks indicating that the density is extremely sensitive to small changes in moisture content. Consequently, contractors find this material very difficult to compact unless the moisture is closely controlled. Through experience, moisture control in silts is directed toward compaction at moisture contents slightly less than the optimum moisture content as determined from the AASHO T-99 laboratory test. However, if the field compactive effort is greater than the laboratory test effort, the optimum moisture content for field compaction should be slightly less than the laboratory value. Thus, the actual field compaction may not be on the dry side of field optimum.

For swelling clays, the general practice appears to be to attempt to compact on the wet side of optimum. This is in conformance with the general awareness that the swelling potential of these soils is considerably less when they are compacted wet of optimum rather than dry of optimum. Unfortunately, the most severely swelling clays are found in the arid regions of the southwest at natural moisture contents much less than the optimum moisture content. In these regions the addition of sufficient water is sometimes impossible and always very costly. Consequently, alternate procedures for reducing swelling potential have been attempted. For example, states such as Texas and Oklahoma appear to be successfully reducing swell problems through the use of lime stabilization techniques. These techniques are beyond the scope of the present paper and will not be discussed here.

In many states not plagued by swelling soils or silty materials, the moisture control is less stringent. In practice, the wet-side control is frequently governed by the mobility of the compaction equipment; i.e., the upper moisture limit is the moisture content at which the compaction equipment begins to bog down. The dry-side control sometimes may be primarily for dust control rather than compaction requirements. The moisture range imposed by these practical considerations usually is too broad to insure satisfactory compaction.

When natural soils are too wet, disking frequently is used to improve the rate of drying through aeration. Disks sometimes are required on jobs where wet cohesive soils are anticipated. In addition, some states encourage construction practices that tend to decrease rewetting due to rainfall during construction. When rainfall is anticipated, the contractors are encouraged to blade a steep crown on the surface and to roll the surface to seal it. This practice increases runoff and frequently eliminates construction delays.

Severe moisture control problems arise in very wet climates such as the Pacific Coast of Oregon and Washington, where the natural soils are very wet and the climatic conditions hinder natural drying. Under such circumstances, it is sometimes impossible to dry cohesive soils satisfactorily, and they must be compacted at moisture contents much higher than optimum. For these conditions, Washington has reported some success with controlling the degree of saturation rather than the moisture and density. Density requirements are reduced so that the degree of saturation does not exceed approximately 87 percent for the minimum moisture contents that can be attained in the field. When this is done the design procedures must be modified to account for the lower strengths to be expected. The concept of adjusting design procedures appears to be a significant one.

To overcome extremely wet conditions, sandwich construction has been successfully utilized for embankment construction in some regions. The wet cohesive soil and coarse granular material are placed in alternate layers. This practice has been successful in regions such as New England where ample sources of granular materials are readily available. However, the procedure becomes impractical when such materials are not plentiful.

When additional water is required, the water is sometimes added in the borrow pit and sometimes on the fill. In general, better moisture control is obtained when the water is added in the borrow pit. This practice is employed both by the Corps of Engineers and the Bureau of Reclamation for earth dam construction, and it is being followed whenever practical for highway construction in many states. However, much highway construction is not well suited to borrow pit operations. Highway construction commonly involves balancing cut and fill sections for which there is no distinct borrow pit. For these conditions the water generally must be added on the fill, but more problems should be anticipated to obtain suitable moisture control and proper mixing on the fill.

Granular Bases

Procedural Requirements for Granular Bases—The size and type of compaction equipment is specified much more frequently for bases and subbases than for embankments and subgrades. Nine states rely entirely on procedural control without density requirements. An additional 22 states specify only procedures for certain classes of work and specify both procedures and density requirements for other work. Most of the minimum equipment requirements in state specifications are related to base course construction.

The greater reliance on equipment and procedural specifications for base courses probably can be attributed to a greater uniformity of base course materials. Select granular materials that satisfy specified gradation requirements are used. Consequently, a satisfactory procedure can be established on a statewide basis.

From the interview program relatively few problems were noted in base course construction. Vibratory compactors frequently are used and satisfactory results are reported. Some states reported problems with degradation of crushed limestone due to compaction, resulting in excessive amounts of fines. Research underway in Iowa appears to indicate that the degradation can be significantly reduced or eliminated through the use of properly adjusted vibratory equipment. Other problems cited were related primarily to difficulties with measuring field densities in coarse materials rather than with the quality of the compacted material.

Maximum allowable lift thicknesses are specified by almost all states. Approximately half of the states specify a maximum compacted thickness of 6 in. The majority of the remaining states specify compacted thicknesses of 3 to 5 in., several specify 8 in., and a few specify 9 to 12 in. In some states, the maximum thickness will vary within the previously indicated range depending on the total thickness and the type of base or subbase. Only 2 states specify loose thicknesses rather than compacted thickness.

Density Requirements for Granular Bases—Forty-one states use density requirements for at least some categories of base and subbase construction. Approximately 10 states base density requirements on the AASHO T-180 test, almost all of which specify 95 percent of the maximum dry density. About 17 states use the AASHO T-99 test for base materials, most specifying 100 percent compaction or greater and a few specifying 95 percent. Fourteen states base density requirements on tests other than AASHO T-99 or T-180. When the AASHO impact tests are not used to establish density requirements, a variety of alternative procedures are used. Most of these attempt to overcome difficulties inherent to application of the AASHO test procedures in coarse materials. Some states express the required density as a percent of the maximum density obtained from a laboratory vibration test. Others express the required density as the percent of a theoretically voidless mass; i.e., the dry density of the solid. For example, Kentucky specifies 84 percent and Wyoming 77 percent of the dry density of solids. Ohio determines base course density requirements on the basis of test sections constructed at each project site. Virginia is also currently trying this technique on an experimental basis.

Most highway engineers appear to feel density requirements for base courses are adequate.

Moisture Control for Granular Bases—Almost 80 percent of the states specify moisture requirements for base courses in a qualitative manner. These statements generally take the form "as required for compaction," or "as directed by the engineer." An additional 5 states specify optimum moisture content, which must be interpreted in a qualitative manner. Five states specify upper and/or lower moisture limits. Two states do not specify moisture control.

Interviews with highway engineers indicate that qualitative moisture control may work more satisfactorily for base materials than for embankments and subgrades. This may be attributed to the select quality of materials utilized for base course construction. Because of the high permeability, the water content cannot be closely controlled; however, for most coarse materials, moisture control is not critical. For dense-graded aggregates, moisture content may be critical but the proper moisture content is easily observable during construction. Consequently, both the contractor and the inspector easily agree on proper moisture conditions.

Other Factors Related to Granular Bases—The most significant factor present in base course construction that is not present in embankments or subgrade construction is the utilization of selected materials. Base course materials usually must meet certain gradation and quality requirements. When natural materials do not meet these requirements, materials are processed to alter the gradation so as to satisfy the specifications.

As a part of gradation requirements, most states specify the maximum percent of fines, usually as the percent passing the No. 100 or 200 sieve. The allowable percentages frequently vary with materials within a state and should depend on the overall gradation of the material. However, approximately 25 percent of the states explicitly require 10 percent or less material finer than the No. 200 sieve. Frequently, the plasticity of the fines also is limited. Occasionally the allowable plasticity index is raised as the percent fines decreases. For example, New York generally permits no more than 10 percent finer than the No. 200 sieve, and the plasticity index of this material must be 3 or less. However, a plasticity index as high as 5 is permitted if the percent of fines is 6 percent or less.

In several states engineers expressed concern regarding particle size degradation caused by compaction. The extent of these effects is generally conceded to be unknown, although as previsously noted research on this subject is underway at Iowa State University. Because of the possibility of degradation during compaction, gradation tests for acceptance of base course materials generally are conducted prior to compaction. This practice permits materials to be accepted that may not meet gradation requirements after compaction and unsatisfactory performance of the base course may result if the percent of fines becomes too large. To avoid such problems, degradation during compaction must be minimized or the maximum allowable percent of fines prior to compaction must be sufficiently small to allow for some increase in fines during compaction.

QUALITY CONTROL PROCEDURES

Quality control and acceptance procedures include density measurements, test rolling and visual inspection. In current practice, most compaction control is accomplished by measuring the field density and comparing it to the maximum density obtainable for this material in a specified laboratory compaction test. Because the control generally is in terms of dry density, the measurement of field moisture content also is involved, even when moisture requirements are not specified quantitatively. Consequently, this section will be concerned with current practices for obtaining field moisture and density measurements and procedures for converting these measurements to relative densities or percent compaction.

The successful implementation of any quality control procedure ultimately depends on the capabilities of the personnel who are responsible for the inspection. Many of the compaction control problems and dilemmas that will be discussed in this section are directly related to the qualifications of the inspection personnel. Consequently, prior to discussing control procedures, the general level of training and experience of earthwork inspectors will be discussed.

Inspection Personnel

The responsibility for earthwork inspection generally resides with the field construction engineer who usually holds an engineering degree and/or has many years of construction experience. However, in practice the actual inspection is performed by an earthwork inspector under the general supervision of the field engineer. Although the background of these inspectors varies considerably within a state, most frequently the inspector is a worker who has little fundamental understanding of the concepts of soil compaction. He may be a new engineering graduate on his first assignment, a college student undertaking summer employment, or a high-school graduate with brief on-the-job training in soil testing procedures. The engineers very quickly advance to more responsible field and office duties, and the better technicians advance to positions as paving and structural inspectors. Thus there is a continual problem of inexperience and of training new personnel. In addition, the wage scales for earthwork inspectors generally are relatively low. Consequently, the position of earthwork inspector is not generally held in high regard, and as a result it is difficult to find competent people to fill these positions.

All of the states visited expressed concern for the problem of obtaining and keeping competent earthwork inspectors. Most states conduct formal or informal training programs for new inspectors. Generally all training programs are oriented toward testing procedures with the trainees being instructed in the following tests: Atterberg limits, sieve analysis, standard laboratory compaction tests and field moisture-density tests. Emphasis is placed on testing techniques and acceptance criteria, and there is little effort to present fundamental concepts of soil behavior.

One of the major dilemmas of compaction control is a result of the qualifications of most earthwork inspectors. Many experienced soils engineers strongly believe that the most satisfactory construction is obtained through visual inspection and the use of engineering judgment with little or no density testing. The attitude is prevalent that much is gained by watching and checking the contractors' operations. The feeling is expressed that when the inspector is performing density tests, the contractor is operating unobserved on another part of the project. These views are undoubtedly valid and correct appraisals of desirable earthwork control when experienced inspectors are available. However, it appears today that the majority of inspectors lack both the experience and the training to make satisfactory engineering judgments that are required by these qualitative control procedures.

Density Control Procedures

Types of Field Moisture and Density Tests—Many state specifications, which include density requirements, do not specify the method by which the field density is to be measured. In many instances, the standard testing procedures are described in a separate manual. In general, no single method dominates current usage. In many states the test methods may vary from one district to another, with the local selection governed by the personal preferences of the district personnel. In some localities, strong opinions exist regarding the relative reliability of the various test methods. Opinions in different parts of the country may be diametrically opposed. The attitudes regarding the reliability of specific tests seem to be related very closely to local experience and details of local testing procedures.

The most common field density tests are destructive tests that involve digging a hole, determining the weight and moisture content of the soil removed, and determining the volume of the hole created. The two most prevalent destructive test methods are the sand-cone method and the balloon method, in which the volume of the hole is determined by refilling the hole with sand and water (in a rubber membrane) respectively. Other techniques encountered occasionally involved refilling the hole with oil, plaster, and, on an experimental basis, paraffin. Approximately the same amount of time is required to perform either the sand cone or the balloon test. The accuracy of these tests is influenced by the coarseness of the soil, with the true volume becoming more difficult to assess when large particles are involved, such as crushed stone or glacial till with rock fragments. To compensate for this, larger holes frequently are used with coarser materials. Most states that encounter such materials have developed rock correction factors with which to adjust test results. The correction procedures differ somewhat from state to state.

The major disadvantage of all destructive density tests is the length of time required to conduct the test, which severely limits the number of tests that can be performed without delays to construction. Often it is not possible to determine whether the material satisfies density requirements until additional lifts of material have been placed over the material that has been tested.

Moisture content determinations usually accompany field density measurements. Because of the time delays involved, the standard laboratory technique of drying for

24 hr in a thermostatically controlled oven generally is not acceptable in the field. Consequently, for many years the prevalent field procedure has been to dry the soils over an open flame. This method, which is still commonly used today, is relatively satisfactory for coarse materials but somewhat unreliable for fine-grained soils. Today many states are using the Speedy Moisture Tester for the field determination of moisture content of fine-grained soils. This relatively new device, which makes use of calcium carbide reaction, permits rapid field moisture determinations. The prevailing opinion regarding experience with this device indicates that it is at least as reliable as the open-flame method which has been used previously. Thus, this device appears to have made a significant contribution to speeding up conventional moisture-density determinations. However, it can accommodate only a relatively small sample and consequently is unsuitable for use with very coarse materials. As a result, states that encounter coarse materials continue to use the open-flame method of drying.

Most states are experimenting with nuclear methods for determining moisture and density. The several available hand portable units are being tested primarily, but some states also are experimenting with the Lane Wells Road Logger, a mobile unit that provides a continuous density record along the length of a fill. The chief advantage of the nuclear methods is the speed of the operation, which permits many more density measurements to be obtained. The major disadvantages are the high cost of equipment and the need for more highly trained technicians. In addition, there remains some concern for the reliability of the nuclear density measurements, although this concern appears to have decreased significantly in recent years. There has been a growing recognition that relatively large variations in density may exist within very small areas of a compacted fill. These local variations are more frequently detected with nuclear devices than with destructive tests. The recognition of these real variations in compacted materials has given impetus to the movement to develop statistical quality control procedures.

In many states, the view was expressed that nuclear methods do not afford as much time saving as is generally reported. While the time required for an individual measurement is much less than by conventional methods, calibration and repair times also must be considered. Most agencies feel that the nuclear equipment must be calibrated for each individual soil and the calibrations must be repeated or checked at least twice a day. Furthermore, perhaps because of excessively rough handling in the field, repairs are required frequently. The repairs, which are made by the manufacturer, are often time consuming and sometimes costly.

As will be discussed in the following section, some states require a standard compaction test, which is conducted in the field using the soil for which the field density has been determined as a part of the field control procedure. In these instances the actual field density determination represents only a small part of the total test time. Consequently, the introduction of nuclear methods produces only a small percent reduction of the total testing time. States that currently use this procedure are hesitant to adopt nuclear methods.

Although the Lane Wells Road Logger can provide more density measurements than the hand portable units, its extremely high cost appears to limit its potential application. Its use does not appear to be economical unless large quantities of earthwork construction were being planned in relatively small geographic areas, perhaps in a large metropolitan area or for an extremely large embankment.

Despite the many problems with nuclear devices, there is every indication that their use will become much more widespread. In the summer of 1965, only Colorado was using nuclear methods for the legal control of earthwork construction. California was beginning a project in each of its eleven districts that would use nuclear control methods. Since that time several additional states have adopted nuclear devices for legal control and it appears fairly certain that additonal states will do so in the future. However, it should be noted that the most effective use of nuclear devices involves more than merely changing the field density determination method. It will probably involve a complete revision of the density specifications to include statistical concepts and conceivably to eliminate the need for the standard laboratory compaction test. The former has already been done, for example, in California where a special specification was written for the jobs to be controlled by nuclear methods.

Field Evaluation of Standard Maximum Compacted Dry Density—In current practice, almost all density requirements are expressed as a percent of the maximum density attained by a specified compaction test procedure. Occasionally, a relative density criterion is used; i.e., the field density requirements are expressed relative to both a maximum and a minimum laboratory density. Thus, to determine the acceptability of the compacted material, the maximum dry density for the material must be established.

From the field interviews, it appears that one of the major problems in the practical interpretation of a density criterion is the proper evaluation of the dry density to which the measured field density should be compared. Typically, laboratory compaction tests are performed on representative samples of primary materials during preliminary planning and prior to construction. The moisture-density or control curves from these tests generally are available for field control. The simplest field control procedure is to compare the measured field density with the control curve that the inspector judges is most representative of the compacted material. To aid in relating control curves to field materials, a library of jar samples of materials for which the control curves were attained is sometimes kept at the job site to facilitate visual identification of materials.

However, in many instances the primary materials are mixed in the earthmoving operations, and as a result, none of the laboratory curves may be directly applicable to the material being placed on the fill. To overcome this problem, most states attempt to make a field evaluation of the maximum dry density in conjunction with field density measurements.

The most common procedures for the field evaluation for maximum dry density involve the use of a one-point field compaction test and/or the development of families of moisture-density curves. One common practice is to use the laboratory compaction test procedure to compact the field material at the placement moisture content. The moisture-density point so obtained is plotted with the family of control curves for the job, and the maximum dry density for the material is estimated by constructing a new moisture-density curve through the test point and roughly similar in shape to the available curves. Some states, Ohio, for example, have developed elaborate statewide collections of typical moisture-density curves. In the Ohio system, the penetration resistance as determined by the Proctor needle is utilized in conjunction with the field compaction test. The statewide family of moisture-density-penetration resistance control curves is then used to estimate the maximum dry density for the field material. A circular slide rule, which is supplied to all inspectors, has been developed to simplify the identification of the proper typical curve from the one-point field data.

These control practices, which require at least a one-point field compaction test in conjunction with each density field test, are time consuming and can potentially delay construction. However, they are deemed necessary in a majority of the states because of the great variations in the materials encountered in highway construction.

As has been noted for the field density test, the laboratory compaction test results also become less reliable when rock-size particles are encountered. In various states, the view was expressed that conventional field test procedures and the laboratory impact compaction test are unsuitable for soils that contain large quantities of rock. A few states, such as Kentucky, have eliminated the need to determine maximum dry density for crushed stone base materials by specifying the density requirements as a percent of the specific gravity of the solids. However, the problems of the measurement of field density remain.

Number and Location of Tests—Until quite recently most states did not specify the number of density tests to be performed or the methods for selecting the position at which the test is conducted, these decisions being left to the judgment of the engineer. During the past few years, however, minimum testing requirements in terms of tests per unit length of roadway or per cubic yard of material placed have been developed in most states. These specifications are being instituted largely at the insistence of the Bureau of Public Roads and frequently over the objections of state highway engineers. Strong opinions are expressed by many highway engineers regarding the unfavorable aspects of specifying the minimum frequency of density testing.

In many states engineers expressed the view that the majority of their compaction work was satisfactorily accomplished without difficulty and, consequently, only minimum results of inspection were necessary. On the other hand, on the relatively few jobs for which problems were encountered, much higher frequencies of testing were deemed necessary. Also, more testing frequently is conducted when a project is first initiated; after the job is running smoothly the rate of testing is frequently reduced. In other words, the states feel that the rate of testing must be related to the degree of difficulty of the particular project. One engineer expressed the view, "80 percent of our testing is done on 20 percent of our compacted materials."

Engineers fear that the minimum frequency of testing will become the standard frequency of testing for all jobs and that, as a result, unnecessary testing will be performed on the satisfactory jobs or insufficient testing will be conducted where problems exist, depending on the established frequency of testing. In most states the tendency has been to set minimum frequency requirements as low as will be accepted by the Bureau of Public Roads. When this is done the states must insure that the inspectors conduct more than the minimum number of tests when problems are encountered.

The selection of locations at which density tests will be conducted has traditionally been left to the judgment of the inspector. Sometimes he is instructed to look for weak spots, on the premise that the density requirements represent a minimum standard which all materials must meet. Also, it is argued that if the weaker-appearing spots can be shown to be acceptable, the entire fill should be satisfactory and the number of tests required can be substantially reduced. Other states instruct inspectors to look for average conditions when selecting a testing site. In this instance it is reasoned that a small weak spot in one lift will not adversely affect the behavior of the completed embankment.

In recent years, random sampling techniques have been proposed for selecting sampling locations. Random sampling is a requirement of the statistical quality control procedures that are currently being advanced for earthwork control. Many highway engineers voice strong objections to random sampling and, as a result, to statistical quality control concepts. Their primary concern is the belief that the statistical procedures will eliminate engineering judgment and increase the chances that an unsatisfactory area will be accepted as a part of the larger section because the random sampling technique did not require tests in the weak zone. They argue that engineering judgment must be maintained to avoid this possibility.

It appears that many of the objections to random sampling and statistical quality control procedures can be eliminated by consideration of the relation of engineering judgment to these procedures. First, before random sampling can be employed, the total area of the section to be considered must be defined. The size of the section is not specified, but rather is selected on the basis of engineering judgment. The section to be considered usually is selected on the basis of uniformity of soil type, moisture content, compactive effort, and other placement conditions. In uniform base course material, several thousand linear feet of material may be considered as a single section, whereas in embankment construction the section more likely may be several hundred feet long because of more frequent changes in material and placement conditions. Hence, the random sampling and the statistical analysis are then applied to the evaluation of the mean quality and the variability of a section that, by visual inspection, the inspector has judged to be uniform. The statistical procedures compensate for the local variability that will exist in even the most closely controlled construction.

The statistical procedures and random sampling are not designed to account for the possibility of a large area within the section that is significantly different from the remainder of the section. For example, in a 500-ft long lift of embankment material a 25-ft long layer of weak material could be present, perhaps as a result of water ponding in a depression during construction. This weak spot might be visually detectable by observing the deformation of this soil under the action of the compaction equipment. However, it is possible that a random-sampling technique based on consideration of the entire 500-ft layer as a uniform section would not designate tests in the weak zone. As a result the unsatisfactory material could be accepted along with the satisfactory material. This possibility illustrates the primary fear of random sampling.

However, this objection can be overcome if the engineer's judgment is properly exercised. In the situation described, proper engineering judgment would dictate dividing the 500 linear feet of embankment into two sections, one of 475 linear feet and the other containing the 25 feet of visually different material. Random sampling and statistical analysis would then be applied independently to each of the two sections, and each would be rejected or accepted separately. Thus, it appears that the proper use of random sampling and statistical control does not eliminate the role of engineering judgment but rather supplements it.

The preceding discussion refers primarily to density testing for control during construction. It should be noted that density testing also is used for documentation purposes after completion of earthwork. Random sampling and statistical quality control are more clearly applicable for this purpose, and few objections were noted to this use.

Compliance with Specifications—With the exception of several recent specifications based on statistical concepts, density requirements are considered to be mimimum standards that must be exceeded by all field test results. If an unsatisfactory test result is obtained, the material is rerolled or removed and replaced, depending on the severity of the deficiency. The material is retested for compliance with specifications. As a result, earthwork construction records will show 100 percent compliance with specifications. However, during interviews with highway engineers, it was admitted that some inspectors do not report unfavorable test results. If, in the inspectors' judgment, an embankment is satisfactory and the majority of test results are acceptable, the inspectors may simply disregard 1 or 2 unfavorable test results. Also, the inspector can affect the test results in marginal cases through his selection of sampling locations. This practice cannot be eliminated without exercising extremely close supervision over inspectors. In fact, the disregard of an occasional unsatisfactory test on the basis of engineering judgment may be a satisfactory and justifiable practice.

Statistical concepts for density requirements have been introduced in part to overcome the problem of the occasional bad test. For any statistical distribution of test results, the probability of an unfavorable test result can only be reduced by raising the mean value or reducing the standard deviation of the test results. Even for closely controlled field experiments the standard deviation can be expected to exceed 2 lb/cu ft (2). Thus, if for illustrative purposes a normal distribution is assumed, approximately 2 to 3 percent of the tests could be expected to fail if the mean value is two standard deviations greater than the minimum requirements. Therefore, for normal construction conditions, an occasional bad test could be anticipated even when the average density of the fill is 5 percent greater than the minimum requirements. The proponents of statistical specifications thus would argue that specifying an allowable percent defective is merely formalizing what is currently informally practiced as a part of engineering judgment.

It appears that the use of statistical compaction control will increase in coming years. However, the acceptance of these procedures by the states will only come with an understanding that statistical methods are a tool to aid engineering judgment and not to eliminate it.

SUMMARY

As a result of the interviews with many highway engineers and the review of current specifications, problems of common concern became apparent. The major difficulties in compaction for highway purposes, as cited by highway engineers, are summarized as follows:

1. Problem soils. The major compaction problems are encountered in construction of embankments of silty soils, swelling clays or extremely wet clays. For these materials, satisfactory placement conditions are difficult to achieve.
2. Heavy construction equipment. The wheel loads of very heavy hauling and paving equipment are overstressing and failing some embankments that would otherwise perform satisfactorily under traffic loads. This problem is most severe for silty embankments.

3. Rapid control procedures. Modern construction equipment and methods have significantly increased the rate of highway construction. As a result, conventional control testing procedures frequently cannot keep pace with construction and delays to construction may result. Most construction engineers are seeking rapid control procedures that will not slow a contractor's progress.

4. Variablility of materials. Because of the variability of natural soil deposits, the raw materials utilized for embankments and subgrades are constantly changing. As a result, most engineers feel that it is neccessary to make field evaluations of the maximum compacted dry density in conjunction with field density measurements. This procedure significantly slows the control testing.

5. Acceptance criteria for materials with high rock content. Materials that contain relatively large percentages of rock fragments frequently are readily placed to form very stable embankments or pavement components. Control and acceptance procedures for the materials commonly are based on the field density, expressed as a percent of standard impact compaction tests. For rocky materials, these tests appear to be slow and inadequate to represent the compaction characteristics of the material.

6. Inspection personnel. In every instance, highway engineers expressed concern for the quality of earthwork inspectors. Although most states have either formal or informal programs for training earthwork inspectors, experienced inspectors are difficult to find. Earthwork inspection is frequently a beginning position from which the more capable individuals quickly advance.

7. Statistical quality control. Statistical quality control procedures and related random sampling plans are causing much concern among state highway engineers. They fear that the role of engineering judgment is being subjugated to handbook statistical methods. However, the inability of the advocates of engineering judgment to provide inspectors who are capable of exercising such judgment has created a vexing dilemma to the states.

Additional problems noted by project personnel but not strongly voiced by highway engineers include the following:

1. Reliance on density criteria. Current compaction specifications are based on the concept that density is a direct indicator of strength. Experimental data from the literature, which are summarized by Langfelder and Nivargikar (3), indicate that strength is affected by many factors in addition to density. Eventually, compaction criteria must be based more directly on strength and other engineering properties of compacted materials.

2. Correlation of laboratory and field compaction. The effects of compaction method on moisture-density relations and the physical properties of compacted materials are noted by Langfelder and Nivargikar (3). There are sufficient data to indicate that the densities and physical properties of samples compacted by laboratory impact methods may differ significantly from the properties of the same material compacted by field construction equipment.

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Factors Influencing the Application of Nuclear Techniques to Soil Compaction Control

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The use of nuclear backscatter moisture-density gages for soil compaction control has gained a great deal of favor during the past several years. However, before this type of equipment can be applied to routine field control, several factors influencing the operation of the gage must be investigated.

Various techniques for expressing nuclear results were studied, and the findings indicate that the use of count ratio at constant high voltage should be adopted. This technique along with the use of standard calibration blocks provides for reproducibility of results and accounts for aging to some extent. These factors are important from the standpoint of recalibrating the gages.

Results of tests performed on various materials and calibration blocks of different chemical composition indicate that material composition has a major effect on the development of calibration curves for the density gages. This was not true for the moisture gages. Soil pH as an indication of soil type was investigated, and calibration curves based on this parameter were developed. The effect of grain size distribution resulted in a different calibration curve for coarse grained vs fine grained soils.

Guidelines for field application were developed. A statistical decision theory based on a t-test was also developed to aid in making a decision involving the validity of using a given calibration curve.

•THE RAPID determination of soil density and moisture content is important in the control of highway construction. Moisture and density control is accomplished by field tests during the construction operation. The speed, accuracy, and reliability of test methods used govern the effectiveness and cost of the control process.

A method of measuring soil density and moisture content using radioactive attenuation has been developed (1). This method was developed on the theory that attenuation of gamma rays can be correlated to soil density because moderation of fast neutrons can be directly associated with the presence of water.

The principal advantages of the nuclear moisture-density instrument lie in its portability, speed of operation, and nondestructiveness. The major shortcomings of conventional methods are eliminated because testing at a specific spot can be accomplished in a matter of minutes. However, use of nuclear gages for final field control has not been universally adopted primarily due to the limited knowledge concerning long-term gage stability and reproducibility, and the fact that the response of the common nuclear density backscatter device is dependent to some degree on the substrate material type.

Paper sponsored by Special Committee on Nuclear Principles and Applications and presented at the 46th Annual Meeting.

PURPOSE

The purposes of this study were to evaluate the nuclear moisture-density technique, and to study the feasibility of using the methods in control of routine highway construction in Indiana. To accomplish this, several commercially available instruments were tested. The variables used as the foundation of the testing program were (a) substrate material properties, (b) instrument stability, and (c) testing procedure factors. No attempt was made to correlate the performance of one instrument to another and, thus, data were selected to illustrate the variables without regard to a specific instrument.

Laboratory work was conducted in the initial phase of the project. The objective of the laboratory work was to provide a basis for establishing testing techniques that were later employed in the field study. Heavy liquids, soils, and soil-aggregate mixtures were tested in the laboratory.

The test sites selected for the field testing phase included natural ground areas, compacted fills, cut areas, loose and compacted subbases, borrow pits and soil waste areas. Material types tested included silts and clays of varying plasticity, granular materials and granular-soil mixtures.

RESULTS

Instrument Stability and Reproducibility

For a nuclear gage to perform satisfactorily as a field control instrument, the reproducibility of results must be consistent over a period of time for a given test variable. If the gage does not perform in this manner, its usefulness and effectiveness may be in doubt.

Test result reproducibility was periodically checked by using voltage plateau curves for the density and moisture gages. Voltage plateau curves were used primarily to provide information concerning the selection of a proper operating high voltage. However, since these curves are self-standard readings plotted as a function of high voltage, the variation of nuclear counts with time is a measure of the gage's ability to record reproducible self-standard readings at constant operating voltage.

Reproducibility of count-ratio results of the density gages were obtained by determining count readings on a concrete block at various high voltages and times. Reproducibility for the moisture gages was determined by self-standard readings.

Each density instrument was assigned a specific concrete block. A permanent outline of the instrument on its block was formed by gluing a piece of weather stripping onto the block. By placing the instrument within the outline and in the same orientation each time a reading was obtained, variations due to placement and direction were eliminated. A count ratio was obtained by dividing the reading on the concrete block by the self-standard reading at the same voltage.

High voltage curves for a density and moisture gage are shown in Figures 1 and 2. Figure 1 shows that for an operating high voltage of 1000 volts (voltage setting employed from June 1962 to completion of testing), the self-standard reading decreased from 43,215 counts per minute (cpm) in January 1962 to 40,469 cpm in July 1964. This represents a decrease far outside the reliable error for the gage. In contrast, the variation of self standards in the moisture gage was within the reliable error (Fig. 2). Figure 1 also shows that from October 1962 to July 1964 the count ratio for the standard concrete block remained nearly constant, varying from 0. 276 to 0. 277 for a high voltage of 1000.

Figure 3 shows variation of the density gage self-standard cpm, count ratio, and moisture gage self-standard cpm with time for constant operating voltage. The data show that the self-standard readings for the density gage with Cs^{137} source decreased with time for constant high voltage. However, for the test period employed, use of the count-ratio procedure eliminated this effect and produced a high degree of test repeatability. The moisture gage with a RaBe source also shows a high degree of reproducibility.

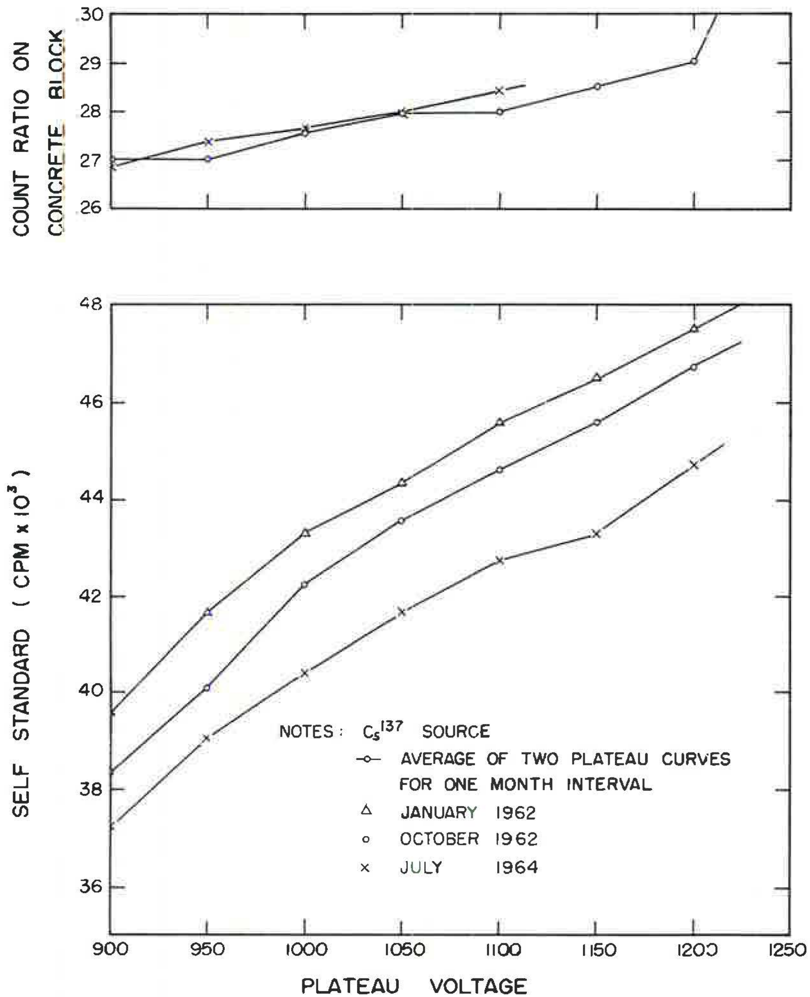

Figure 1. Variation of density gage readings with plateau voltage.

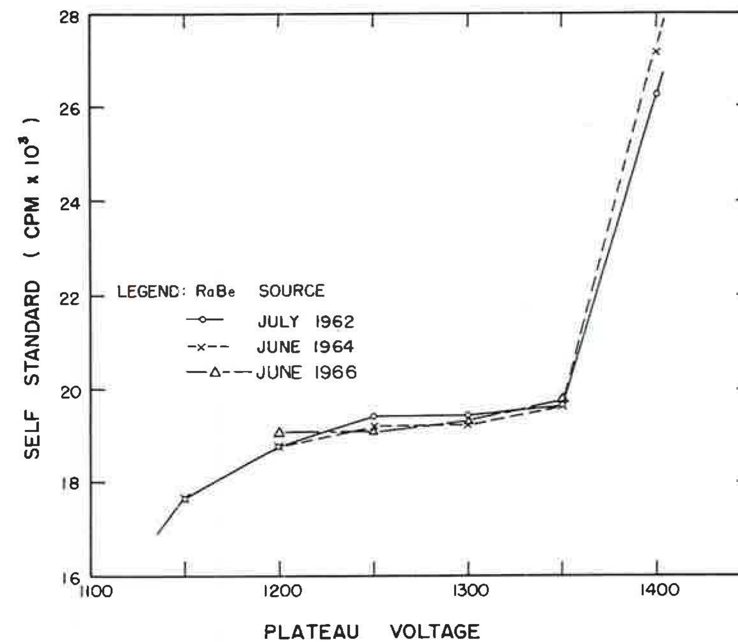

Figure 2. Variation of moisture gage readings with plateau voltage.

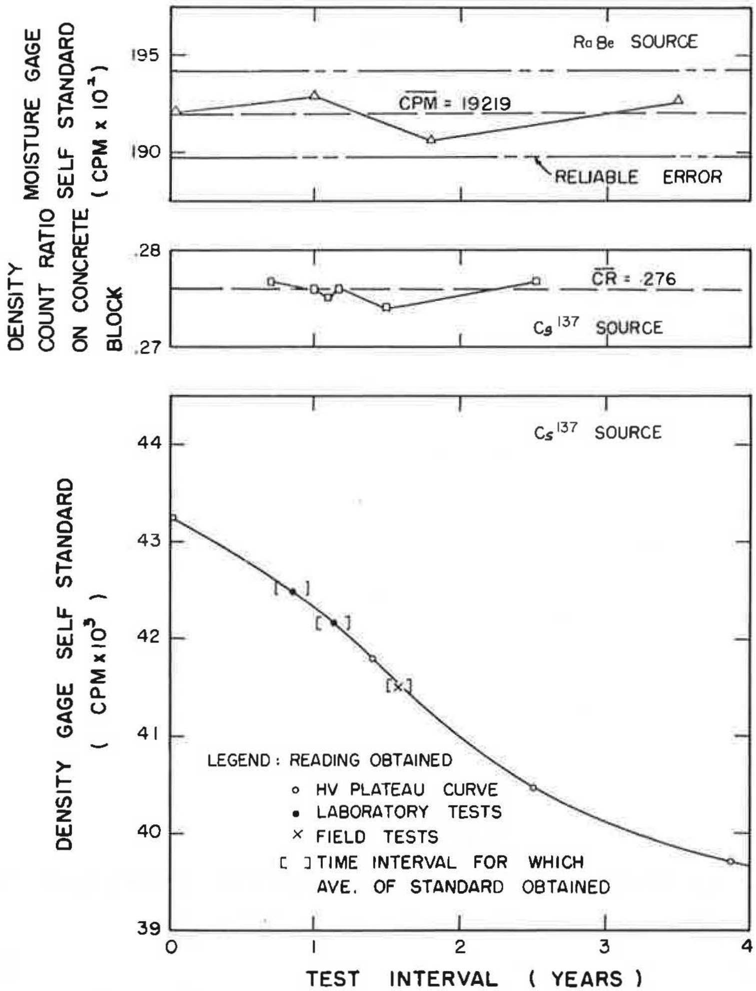

Figure 3. Variation of gage readings with time at constant plateau voltage.

Aging

Radioactive materials decay at differing rates depending upon the half-life of the source. The decay rate is governed by the natural radioactive decay law and, consequently, the reproducibility of self-standard readings for nuclear gages is a function of the radioactive source employed.

The half-life of the Cs^{137} source shown in Figures 1 and 3 is 33 yr as opposed to the 1620-yr half-life of the RaBe source shown in Figures 2 and 3. Therefore, a nuclear gage utilizing a Cs^{137} source would obviously show a larger proportional decrease in counts measured by a detector tube than a nuclear gage using a RaBe source for a given time interval.

If decay ratio is defined as the nuclear activity at time zero to the activity at time (t), theoretical and actual decay ratios can be computed and compared for the density gage utilizing the Cs^{137} source (Fig. 3, bottom). The general equations for radioactive decay are

$$N(t) = N_0 e^{-\lambda t} \tag{1}$$

$$A(t) = \lambda N(t) \tag{2}$$

$$T = \frac{\ln 2}{\lambda} \tag{3}$$

where

$N(t)$ = number of undecayed atoms at time (t),
N_0 = number of undecayed atoms at time ($t = 0$),
$A(t)$ = activity at time (t),
A_0 = activity at time ($t = 0$),
t = time from $t = 0$ (yr),
λ = proportionality constant (yr^{-1}) = 0.021 for Cs^{137}, and
T = half-life (yr) = 33 yr for Cs^{137}.

Therefore, the theoretical decay ratio becomes

$$R_T = \frac{A_0\,(t = 0)}{A_t\,(t = t)} = \frac{\lambda N_0 e^{-\lambda\,(t = 0)}}{\lambda N_0 e^{-\lambda t}} = \frac{\lambda N_0}{\lambda N_0 e^{-\lambda t}}$$

$$= \frac{1}{e^{-\lambda t}}$$

$$= e^{\lambda t} \tag{4}$$

TABLE 1

COMPARISON OF ACTUAL AND THEORETICAL DECAY RATIOS

t (yr)	$e^{\lambda t}$	R_T	$\frac{CPM\ (t = 0)}{CPM\ (t = t)}$	R_A
0 (Jan. 1962)	$e^{(0.021)\,0}$	1.000	$\frac{43{,}215}{43{,}215}$	1.000
0.83 (Oct. 1962)	$e^{(0.021)\,(0.83)}$	1.018	$\frac{43{,}215}{42{,}531}$	1.016
1.17 (Feb. 1963)	$e^{(0.021)\,(1.17)}$	1.025	$\frac{43{,}215}{42.181}$	1.025
1.50 (June 1963)	$e^{(0.021)\,(1.50)}$	1.032	$\frac{43{,}215}{41{,}880}$	1.032
1.67 (Aug. 1963)	$e^{(0.021)\,(1.67)}$	1.036	$\frac{43{,}215}{41{,}544}$	1.040
2.58 (July 1964)	$e^{(0.021)\,(2.58)}$	1.056	$\frac{43{,}215}{40{,}469}$	1.068
3.92 (Nov. 1965)	$e^{(0.021)\,(3.92)}$	1.086	$\frac{43{,}215}{39{,}696}$	1.089

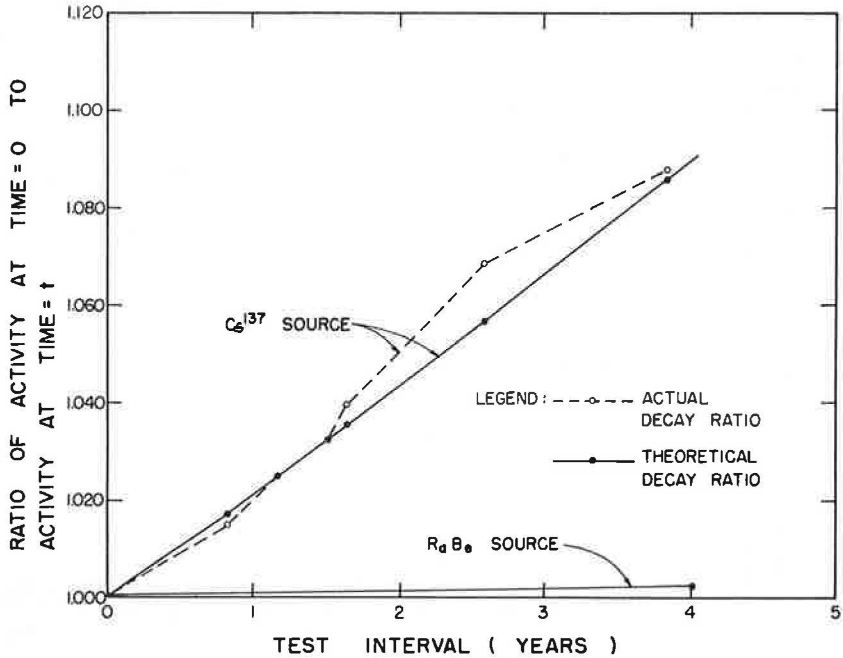

Figure 4. Comparison of actual and theoretical decay ratios.

For the actual decay ratio

$$RA = \frac{\text{CPM } (t = 0)}{\text{CPM } (t = t)} \tag{5}$$

Table 1 and Figure 4 show a comparison of the actual and theoretical decay ratios as a function of time for the Cs^{137} source. From these it appears that the decrease in self-standard counts for the density gage given in Figure 3 can be attributed to decay of the source. For the RaBe source the theoretical decay ratio at time ($t = 4$ yr) would be $e^{(0.00043)4} = 1.00174$. This value is also plotted in Figure 4. As mentioned previously, the decrease in self-standard counts for the moisture gage utilizing the RaBe source was almost negligible.

Although the self-standard readings for the density gage decrease due to decay, the use of a count ratio tends to correct for the decay. Pocock (10) has shown mathematically that the use of a count-ratio procedure will not completely eliminate variations due to source deterioration.

In his paper Pocock states:

> It becomes apparent that use of the count-in-soil to count-in-standard ratio will not eliminate the effect on the calibration curve of half-life in reducing source strength in practice. Yet, although use of the ratio will not eliminate the effect of half-life, it is possible that its use may reduce this effect.

He further states:

> It appears...that use of the ratio, for the purpose of lessening the effect on the calibration curve of reduction of source strength with time as a function of half-life is justifiable on theoretical grounds. It should be emphasized that use of the ratio will not eliminate the need for recalibration, but will merely serve to reduce the required frequency of recalibration.

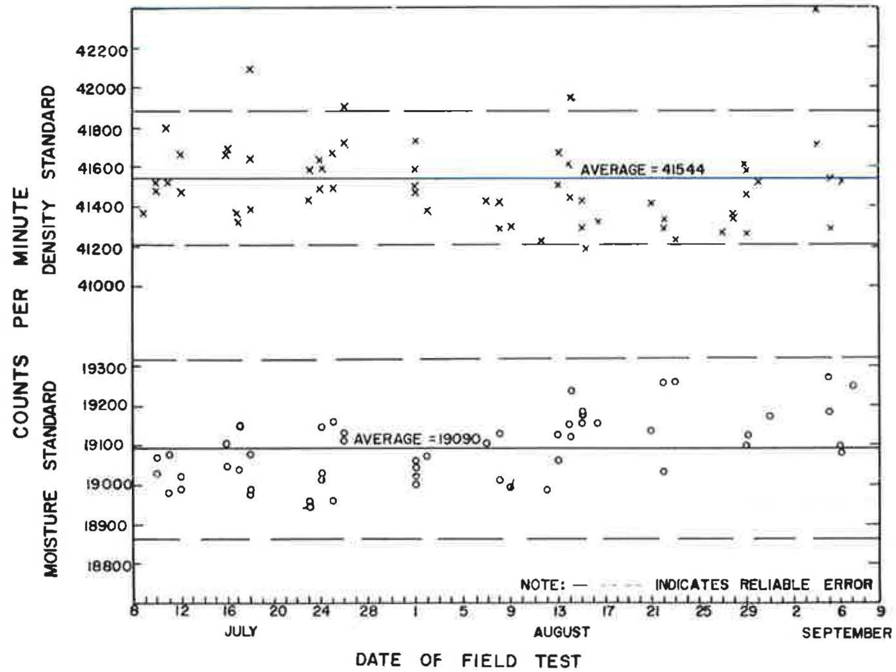

Figure 5. Variability of standard counts.

Interpreting the data obtained from Figure 3 along with Pocock's discussion, it can be concluded that for a testing period of approximately 2.5 yr the use of the count ratio eliminated the need to recalibrate the nuclear gage containing the Cs^{137} source. It cannot, however, be concluded, because of the limited test interval, that recalibration will never be required when the count ratio is utilized.

The previous paragraphs have been concerned with the effect of aging on nuclear results over extended periods of time. For short periods of time (several months) aging does not appear to affect the standard counts for either the moisture or density gages (Fig. 5). The density gage for the particular instrument system indicated utilizes a Cs^{137} source which is the most critical with respect to aging. However, results as shown in Figure 5 indicate that standard counts remain constant over relatively short periods of time.

Procedural Factors

Count Ratio vs Counts per Minute—Nuclear readings can either be expressed as cpm or as a count ratio (relative count). However, since the value of the nuclear reading is also a function of the operating high voltage value of the instrument, data may be reported using either a constant or variable high voltage procedure.

Figure 6 illustrates this situation based on the data for Figure 1. The dotted line represents the anticipated plateau curve at a time (t'') when the self-standard count (C''_U), at a high voltage setting equal to the upper limit high voltage (HV = U), is identical to the initial self-standard count (C_A) obtained at a high voltage setting HV = A.

Three methods of expressing the results are shown in Figure 6. The procedure that utilizes results expressed as cpm is attained by varying the high voltage to maintain the original self-standard counts. Thus, if C_A represents the initial self standard obtained at time (t_0) and C'_B represents the same self-standard reading at time (t'), the high voltage would have to be varied from HV = A to HV = B. The particular nuclear count obtained on any substrate material would then be the result expressed in cpm.

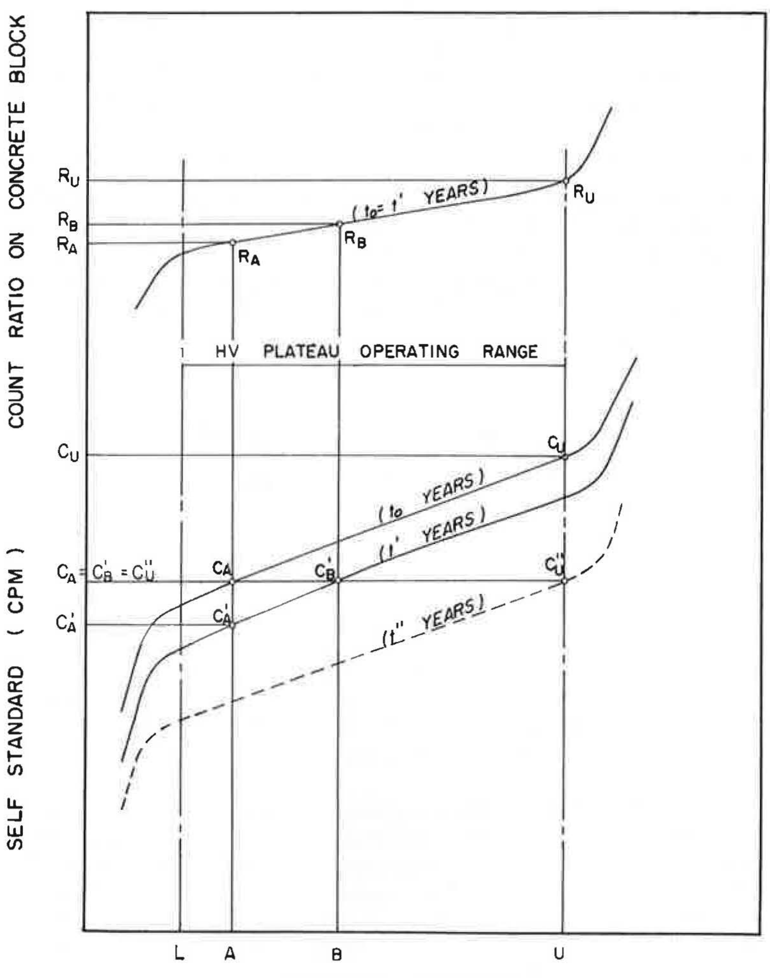

Figure 6. Illustration of density gage readings with plateau voltage.

If a count-ratio procedure utilizing variable high voltage is used, the second procedure is defined. As the voltage is varied from HV = A to HV = B, the count ratio on the standard block at time (t′) would change from R_A to R_B.

The count ratio of the standard block for the third method is obtained by keeping the operating high voltage constant. If the initial count ratio for the standard block at time (t_o) is R_A, the count ratio at time (t′) would also be R_A.

Based on these data, the best procedure to employ when using a nuclear gage is to express the results in the form of a count ratio obtained by keeping the operating high voltage constant at any given time. This is explained in the following manner.

If a count-per-minute method is used with variable voltage to achieve the initial self-standard reading, three facts are noted.

1. Although the self-standard reading has been kept constant ($C_A = C'_B = C''_U$), the reading obtained on a substrate material (in this case a standard concrete block) will increase as the high voltage is varied with time to obtain the initial self-standard reading. Note that the count-ratio (R) curve on the standard block increases with high voltage setting but does not change as a function of time.

2. There will be a definite time period when recalibration will be necessary due to (a) a change in the standard block reading, from time (t_0) to (t'), becoming equal to or greater than the upper reliable error for the count of the standard block reading obtained at time (t_0), or (b) an increase in high voltage that results in a voltage greater than the upper limit of the plateau operating range (HV = U).

3. Use of cpm will not eliminate any variations due to instrument instability.

For the count-ratio procedure also using a varying voltage, the results are identical to the cpm procedure described, with the exception that the count-ratio procedure will eliminate several effects of instrument stability and physical surrounding. However, since the high voltage is variable, the use of the count ratio in this procedure (variable high voltage) will not eliminate any effects of time (aging). This is illustrated by the following:

When

$$t = t_0\text{: HV} = A\text{, count per minute} = C_A\text{; count ratio} = R_A$$

$$t = t' \quad \text{HV} = B\text{, count per minute} = C'_B\text{; count ratio} = R_B$$

However, from Figure 6

$$R_A \neq R_B$$

The operating high voltage is a function of time as the high voltage is varied with time to obtain a constant self-standard reading. The time required for recalibration can be measured in terms of the high voltage necessary to produce the two cases previously mentioned knowing only the self-standard and count-ratio plateau curves at time (t_0).

It is assumed that the recalibration curve will be parallel to the original calibration curve (i.e., the slope of the count ratio curve for the standard block, a discrete density value, would be equal to slopes of all count-ratio curves obtained in a similar manner at any given density). Although data were not obtained for various standard block densities, the slope of the curves is dependent only on the electronic system used in the particular gage. Therefore, for a given nuclear gage, the assumption of parallel recalibration curves seems valid.

For the data obtained in Figure 1, the high voltage at which the count ratio of the standard block was equal to the initial count ratio plus the upper reliable error was HV = B = 1110 volts. This corresponded to a time of approximately 1.8 yr. In other words, if the count ratio procedure with a variable voltage had been used, the gage would have had to be recalibrated 1.8 yr from the date testing was initiated.

The upper limit of the plateau HV for the gage tested was 1200 volts. Consequently, once the operating high voltage had been varied from the initial operating voltage (HV = A = 1000 volts) to the upper limit high voltage (HV = U = 1200 volts) the gage would also have to be recalibrated. An approximate time for recalibration can be found by equating the actual decay ratio (at HV = 1200 volts; $C_A = C_U' = 43{,}215$; and $C_U = 47{,}500$) to the theoretical decay ratio $e^{\lambda t}$ for a Cs^{137} source:

$$e^{\lambda t} = \frac{C_U}{C_U} \tag{6}$$

$$e^{(0.021)t} = \frac{47{,}500}{43{,}215}$$

$$e^{(0.021)t} = 1.099$$

$$t = \frac{\ln_e 1.099}{(0.021)}$$

$$t = \frac{0.0943}{0.021}$$

$$t = 4.5 \text{ yr}$$

Therefore, if a count ratio with a varying high voltage procedure was used to express nuclear readings, it would take approximately 4.5 yr for the high voltage setting to reach the upper limit of the plateau operating voltage (HV = U) for the data presented in Figure 1.

For the count-ratio procedure using a constant high voltage at any given time the following results are stated. The count ratio obtained on a standard concrete block remained constant for approximately 2.5 yr. Thus, for 2.5 yr the use of the count ratio not only eliminated the effects of variations in readings caused by instrument stability (instability), it also eliminated the effects of aging due to source decay. Figure 7 shows the effect of data scatter reduction obtained on a nuclear moisture gage employing a RaBe source. As the RaBe source has been shown to have a negligible effect on nuclear reading variations on a standard system over a period of time due to its long half-life (see Fig. 2), the scatter reduction can be attributed mainly to variations of instrument stability factors.

However, as previously mentioned, it cannot be conclusively stated, because of the limited time interval, that the use of a count ratio will completely eliminate the effect of source decay on nuclear readings. The important concept is that a count-ratio procedure used at a constant high voltage will require recalibration at less frequent intervals than if a count ratio using a varying-voltage procedure is utilized.

Consequently, if it is assumed that the life of the instrument's electronic system will be more than 1.8 yr, recalibration of nuclear instruments using a Cs^{137} source will require recalibration due to source decay (aging), provided the method used to express results is a count-ratio-varying high voltage.

If a count-ratio-constant high voltage procedure is used, no definite time required for recalibration can be made because the data obtained indicate a constant count ratio

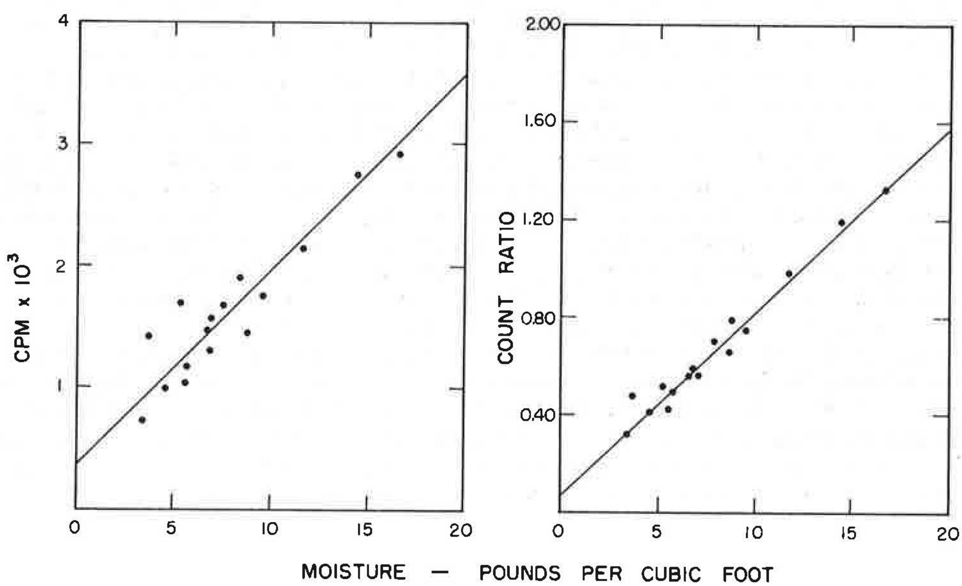

Figure 7. Comparison of count ratio and cpm procedure for moisture calibration.

on the standard block for 2.5 yr. However, if it is assumed that the argument presented by Pocock (10) is correct concerning the fact that the count ratio will not completely eliminate the effect of aging, then the statistic of importance depends on the time required for the count ratio, at a constant high voltage (R_A), to change to R_A' where $R_A - R_A' >$ reliable error of (R_A). Consequently, since this difference in count ratios was negligible for a 2.5-yr period, the time required to cause a recalibration (count-ratio-constant high voltage) for a nuclear gage using a Cs^{137} source might be far longer than the time required for a recalibration necessitated by an electronic failure. Therefore, it is felt the primary reason for recalibrating a nuclear gage containing a Cs^{137} source will be due primarily to electronic failure, provided a count-ratio-constant high voltage procedure is used.

The general results and procedures stated can be adapted to any nuclear gage (density or moisture) using any nuclear source. However, it must be emphasized that the relative magnitude of these results is a function of the source (half-life) used in the nuclear gage.

SUBSTRATE MATERIALS

Density Gage and Material Composition

The most important item of conjecture in the application of nuclear density gages to field use has been the influence of material type upon density readings. By using a single calibration curve, or assuming that all material types will respond identically to the substrate system at a given density, the assumption of equal mass absorption coefficients for all material types is made. However, mass absorption coefficients are a function of both the nuclear particle energy and the type of element. Values of mass absorption coefficients and their dependence upon elements commonly found in soils have been given by Parsons and Lewis (7) and are shown in Figure 8.

At energy ranges higher than about 0.3 Mev, the absorption coefficients for all elements shown, with the exception of hydrogen, are relatively constant. Conversely, wide variations between coefficients are evident for energy ranges less than 0.3 Mev. Figure 9 shows calibration curves for heavy liquids, a crushed limestone and a crushed quartzite. Both crushed materials had identical grain size distribution curves graded to $p = 100\ (d/D)^n$ where $D = {}^3\!/_8$ in. and $n = 0.5$.

In July 1965 the nuclear gages were taken to Charlottesville, Virginia, for the Correlation and Conference of Portable Nuclear Density and Moisture Systems conducted by the Virginia Highway Research Council. The nuclear density gages were calibrated on a series of calibration blocks for the Virginia study. The chemical analysis of each of these blocks, along with a chemical analysis of the crushed limestone and crushed quartzite studied at Purdue, is given in Table 2. Blocks 1, 2, and 5 have an appreciable quantity of silicon dioxide (SiO_2) and are similar in chemical composition to the crushed quartzite studied. Similarly, blocks 3 and 4 are similar to the crushed limestone in that the predominant chemical compound is calcium oxide (CaO). It is obvious that for the SiO_2 blocks the nuclear readings are near the calibration curve established for the crushed quartzite obtained in the laboratory investigation. A similar relationship exists between the CaO blocks and the crushed limestone curve.

Figure 9 shows the effect of material type for nuclear density gage with a Cs^{137} source. For the nuclear gage with the RaBe source, identical patterns for the crushed materials and calibration blocks were noted with the exception that a larger deviation between the limestone and quartzite curves occurred.[1] It is felt that a possible explanation for this event is directly related to the type of source used in each gage.

RaBe has the major portion of its energy spectra at two energy levels, 0.61 Mev and 0.35 Mev. Since a portion of the initial energy is lost due to the physical events that occur in the system, and there also exists some radiation at energies of 0.18 Mev at

[1] For the gage using a Cs^{137} source, the magnitude of the deviation ranged from 12 pcf to 18 pcf, while the range between calibration curves was 25 pcf to 35 pcf with the gage using the RaBe source.

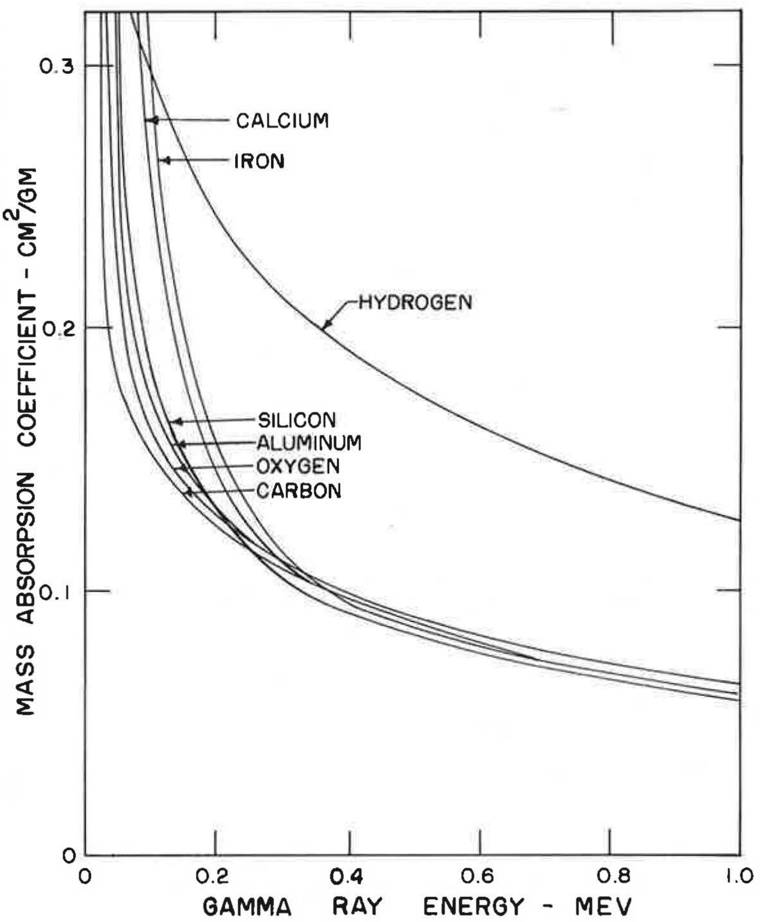

Figure 8. Relations between mass absorption coefficient and the energy of gamma radiation for elements commonly found in soil (from Parsons & Lewis).

the lower spectrum value, it is suggested that the possibility for radiation levels being found at or below the 0.3 Mev energy range is quite probable.

TABLE 2

CHEMICAL ANALYSIS OF SELECTED MATERIALS

Material	SiO_2 (%)	CaO (%)	Other Chemicals (%)
Block No. 1 (Virginia)	100	—	—
2 (Virginia)	74.2	—	25.8
3 (Virginia)	—	54.0	46.0
4 (Virginia)	—	55.8	44.2
5 (Virginia)	74.4	—	25.6
Crushed quartzite	97.2	—	2.8
Crushed limestone	12.1	47.1	40.8

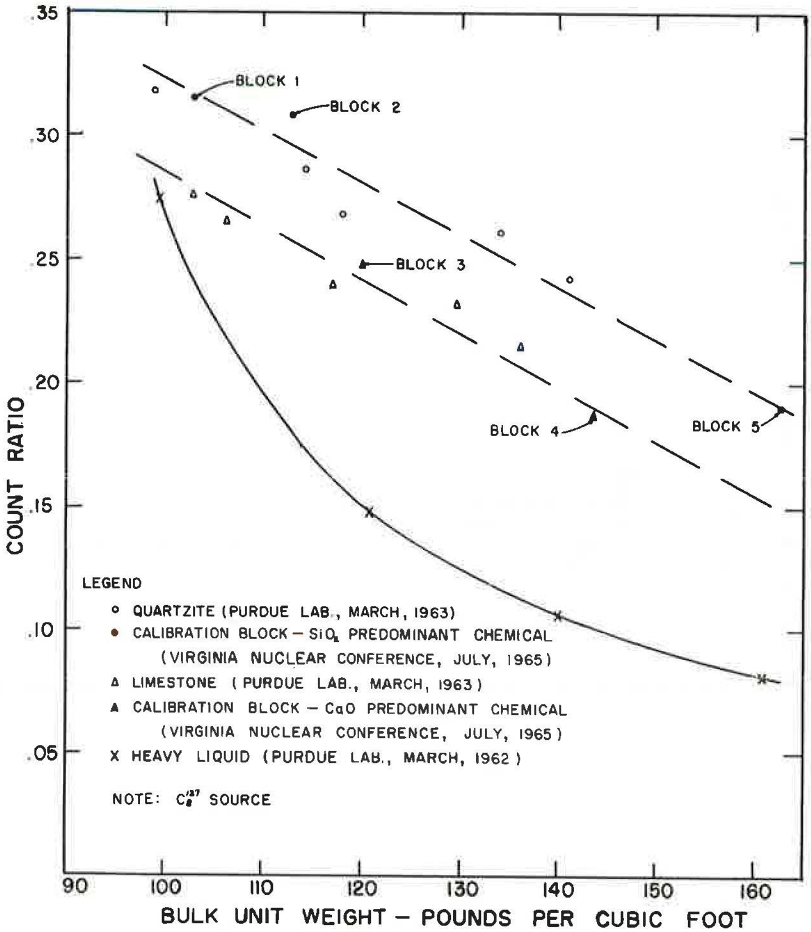

Figure 9. Effect of material type on nuclear density readings.

Since a Cs^{137} radiation source has an energy level in which the initial level of the photon energy exists at 0.66 Mev, a subsequent smaller portion of photon energies may be expected to occur at the 0.3 Mev level. Therefore, smaller deviations between chemically different soils may be expected to occur for a Cs^{137} source due to its radiation energy spectrum existing at energy levels where mass absorption coefficients for most soil elements are almost identical.

For energy levels below 0.3 Mev, a rather large deviation in mass absorption coefficients occurs between calcium and silicon (Fig. 8). Because of this, the concept of using soil pH as an indicator to correlate the mass absorption was used as a field experiment to determine material type.

It is recognized that perhaps the pH method can only be used in a general way to indicate material type because it is possible for a large proportion of an element in a soil to produce a weak acid while a small proportion of another element may produce a strong base. The titrating effect of the weak acid and strong base solution may tend to yield a basic pH while physically speaking, the acidic element would generally dominate the overall average mass absorption coefficient for the soil. Also, soluble salts in the soil mass of different chemical properties than that of the soil itself may result in the measurement of a pH value that is not truly indicative of the soil. However, field tests were conducted using pH as an indicator because of its relative ease of use in the field in contrast to a more complicated procedure of obtaining a quantitative analysis of soil composition.

Figure 10 shows the results of nuclear density tests conducted in the field. All densities were obtained by the sand-cone method. Two distinct calibration curves were developed; one for basic soils and one for acidic soils. Assuming that a basic soil would generally correspond to those soils containing large quantities of calcium and iron, and acidic soils would correspond to elements shown in Figure 8 that are distinct from the calcium and iron for energy levels below 0.3 Mev, it would be expected that a basic soil would absorb more of the nuclear particles than an acidic soil (i.e., fewer counts would be recorded and a basic soil calibration curve would then plot below an acidic soil calibration curve). The general relationship of the basic and acidic curves for data in Figure 10 tend to verify this concept.

The standard error of estimate for the basic soil curve was ± 0.010 (count ratio) and ± 0.015 (count ratio) for the acidic soil curve. The overall standard error of estimate for all data regardless of pH was ± 0.017 (count ratio). Based on these results, it can be stated that, although use of pH cannot be completely correlated to mass absorption coefficients (i.e., material type), its application to field testing produced calibration curve parameters that reduce errors of estimate for the nuclear readings.

Grain Size Distribution

Reference has been made to the importance of mass absorption coefficients of various soil elements for nuclear determination. However, for a particular material

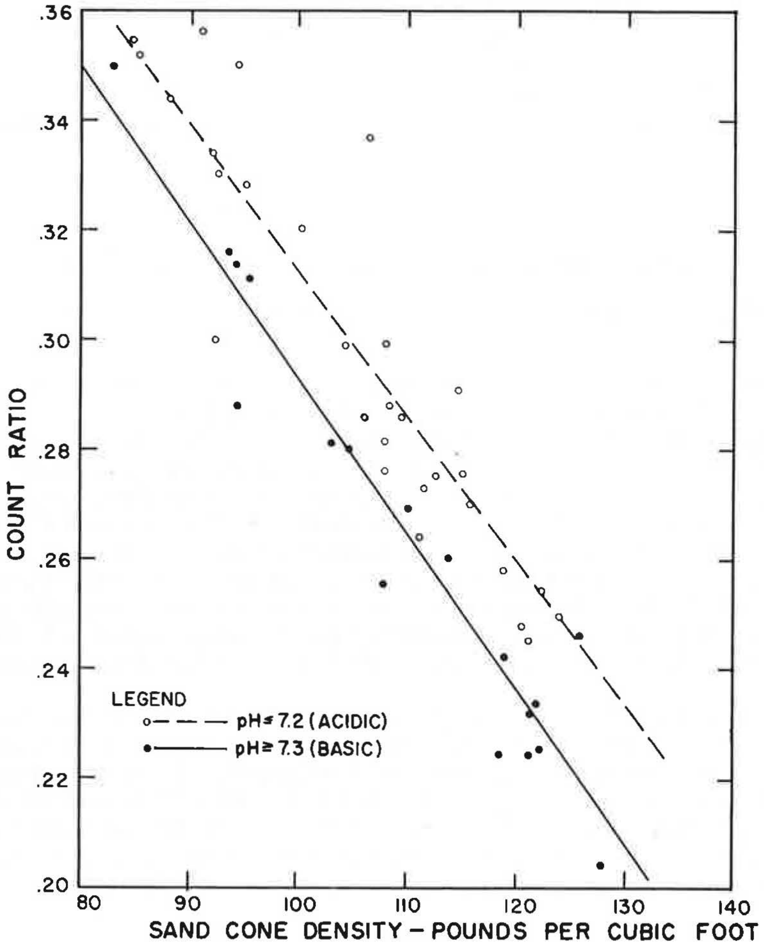

Figure 10. Density calibration curves developed by soil pH grouping for field data.

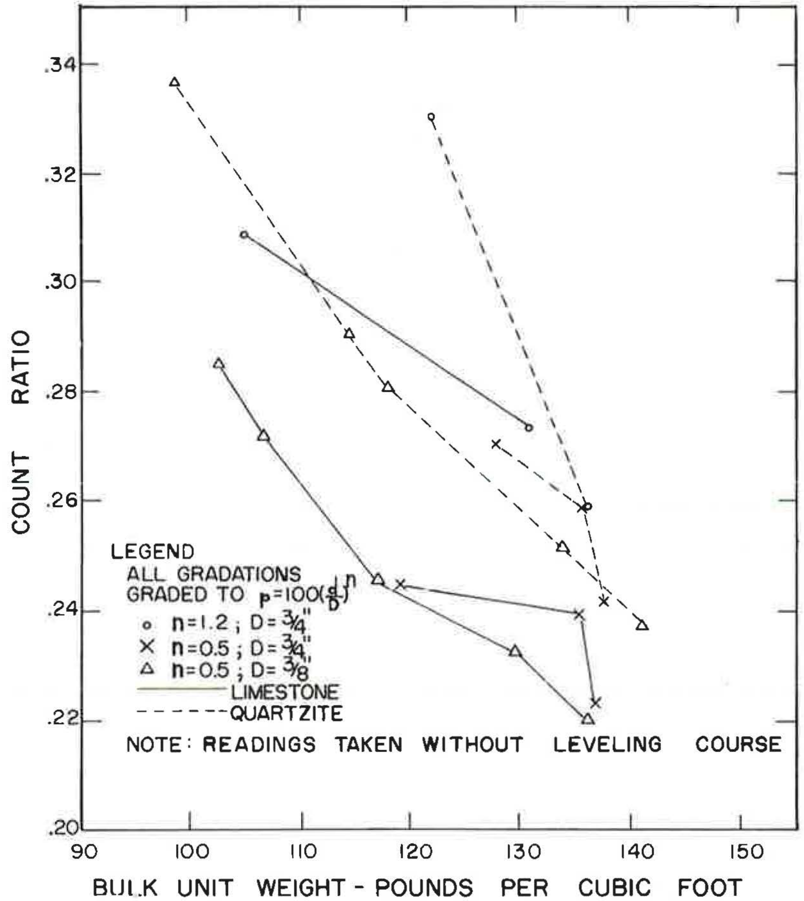

Figure 11. Effect of grain size distribution on nuclear density readings for selected materials.

type, the question arises, can similar mass absorption coefficients be defined at various compositions of the soil, or at different grain size distributions?

To investigate this effect, two materials were tested. They were subsequently crushed and hand picked to produce a finer (denser) gradation. For a given gradation, each material was blended to yield identical grain size distributions (Fig. 11).

Both instruments tested produced similar count reductions for both the quartzite and limestone materials as the open graded material was crushed finer. From the results shown, it was concluded that the nuclear gage did not "record" identical mass absorption coefficients for the same material at the grain size distribution indicated in Figure 11. Field data tend to substantiate this concept as shown in Figure 12. The figure is representative of all basic (pH ≥ 7.3) field materials plotted as granular vs fine grained.

However, it is not felt that for every possible grain size distribution for a given material type, deviations between calibration curves can be expected to exist. Rather, it is felt that at a certain state of grain size distribution, this effect is negligible. The data are indicative only for aggregates and soil aggregate mixes and are limited in quantity, preventing expression of a general conclusion. However, in studies made during July 1966 this same trend was observed using a different nuclear system, and it would appear that distinct parallel calibration curves exist for coarse grained vs fine grained materials.

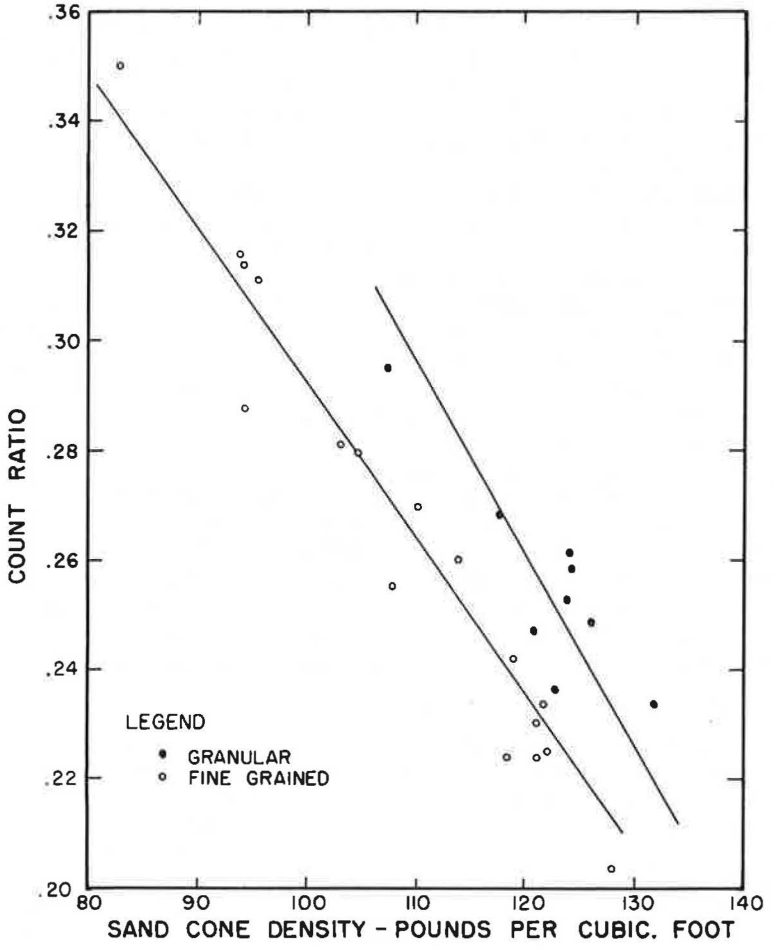

Figure 12. Comparison of field calibration curves for granular and fine grained materials (pH ≥ 7.3).

SUMMARY

Guidelines for Use of Nuclear Gages

This section presents guidelines for the use of nuclear moisture density gages for routine compaction control in Indiana. The following procedures are based on experiences gained in this study. It is felt that by following the recommended format, reliable results can be obtained with these instruments. Note that these recommendations are not considered to be the ultimate, but represent the best techniques developed up to the present time. Furthermore, the following general procedures apply to all nuclear gages, regardless of manufacturer.

I. General Concepts Involved in the Use of Nuclear Gages

A. Method of Reporting Results—The use of a count-ratio procedure at constant voltage should be adopted for expressing all nuclear counts for the gages.

B. Standard—To achieve more consistent readings, the use of an air gap to obtain the instrument standard readings is recommended. The same air gap device can be used with all gages and it need not be elaborate in construction. A simple wooden platform approximately 12 in. in height has been found to be highly satisfactory.

C. Leveling Course—It is highly desirable to obtain a flat surface on the test area in order to insure proper seating of the gage. Many researchers have suggested the use of a leveling course to be placed between the gage and substrate to accomplish this seating. However, results during this study did not conclusively substantiate this concept and no recommendation concerning this factor can be made.

D. Standard Blocks—A standard reference block should be used to insure proper functioning of the density gage as well as to check reproducibility of results. The blocks can also be used to establish count ratio plateau curves for the gages. Three conditions should be met: (a) each gage should have its own individual block, (b) the blocks should be made of a material which will not change density or chemical composition with time, and (c) the gage should always be placed on this block in the same orientation. These conditions are necessary to insure that the gage is influenced by the same volume of material for all readings. As the block does not need to be homogeneous in order to serve as a standard, it is suggested that concrete blocks of not less than 24 by 24 by 12 in. be made for each gage. These blocks should be stored in some central location where periodic check tests can be performed. These tests should be performed at least every 3 months. When the count ratio of the standard block at a given voltage is outside the established reliable error, it will be necessary to recalibrate the gage. This is especially critical for gages utilizing sources which have a relatively short half-life. Any adjustments or changes made on the equipment by the manufacturer will also necessitate checking the gage to determine if recalibration is warranted.

II. Moisture Gages

A. Calibration Curves—It is recommended that the laboratory moisture calibration curves be adopted for field use. A typical moisture calibration curve is shown in Figure 13. If the gage is to be utilized on granular materials at low moisture contents, a comparison of the expected depth of penetration and depth of the granular material should be made. In using the moisture gages the following procedure is recommended.

1. On each project several check tests should be performed to insure that the calibration curve is valid for that project before actually using the data for moisture control. The check tests may be made by following these steps.
   a. Test sites on typical soils should be prepared by providing a smooth level surface on which to place the nuclear gage. Nuclear counts should be taken on the test area and an average count ratio determined.
   b. A sand-cone density test should then be performed on the exact area where the nuclear gage had been placed. The sand-cone density obtained is used to determine the pounds of water per cubic foot of soil, and to serve as density gage check tests.
   c. The moisture content of the material taken from the density hole should be determined by standard laboratory ovendry techniques. If this cannot be accomplished, field moisture determinations should be made using one of several techniques now in use.
   d. After obtaining values for the dry density and moisture content in percent, the moisture data should be converted to pounds of water per cubic foot. This value should then be plotted with its corresponding nuclear count ratio on the laboratory calibration curve.
2. The required number of check tests should be determined from Figure 16 for the particular Type II error (β) desired.
3. The suggested format to be used in either accepting or rejecting the moisture calibration curve based on the results of the check tests is shown in the Appendix.

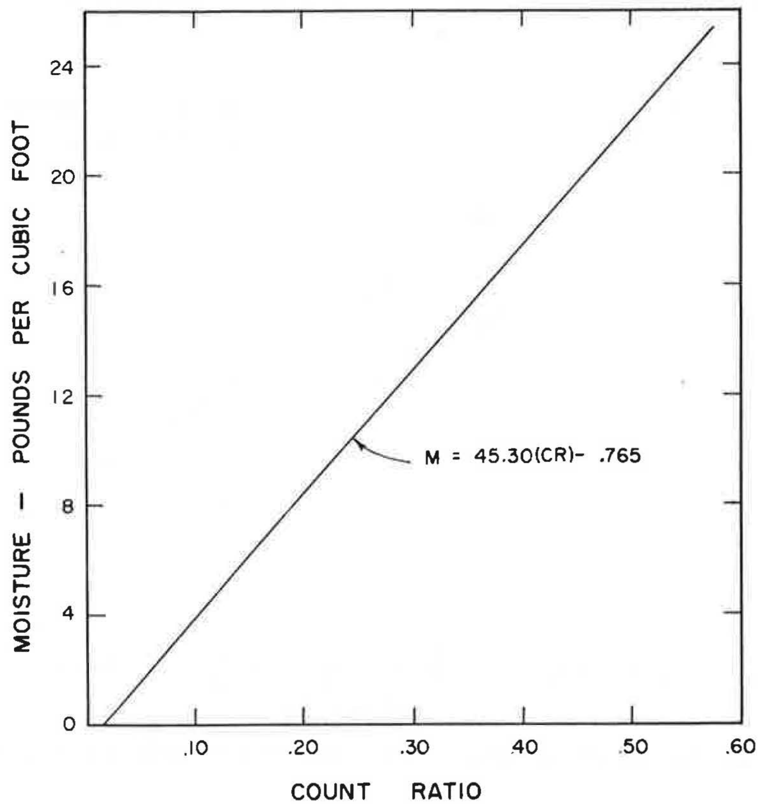

Figure 13. Moisture calibration curve for instrument No. 1.

III. Density Gages

A. Calibration Curves—Typical density calibration curves are shown in Figures 14 and 15. These curves were obtained under field conditions and are recommended for use in field compaction control. The curves are based on two primary types of soils: (a) subgrade or embankment soils and (b) subbase materials. The first category is further subdivided on the basis of soil pH into a basic and acidic classification. In order to utilize the density gage, the following procedure is suggested.

1. On a given project, several check tests should be performed on the typical soils involved. In making these tests, a procedure similar to that described for the moisture gages should be followed (as pertains to obtaining a flat surface, etc.). It is necessary to obtain an average count ratio and a sand-cone density for each check test. Also, for the subgrade or embankment soils, it is necessary to measure the pH of the soil. This measurement can be made by either using a portable, battery operated pH meter or by using soil color charts. Both of these tests are simple to perform and the equipment involved is relatively inexpensive.
2. The required number of tests necessary to ascertain the validity of a given calibration curve as well as a suggested format that allows a statistical decision to accept or reject the curve are presented in the Appendix.
3. If the decision to reject the calibration curve is made, a new calibration curve should be developed. This is accomplished by performing further tests on the construction materials and plotting the data as

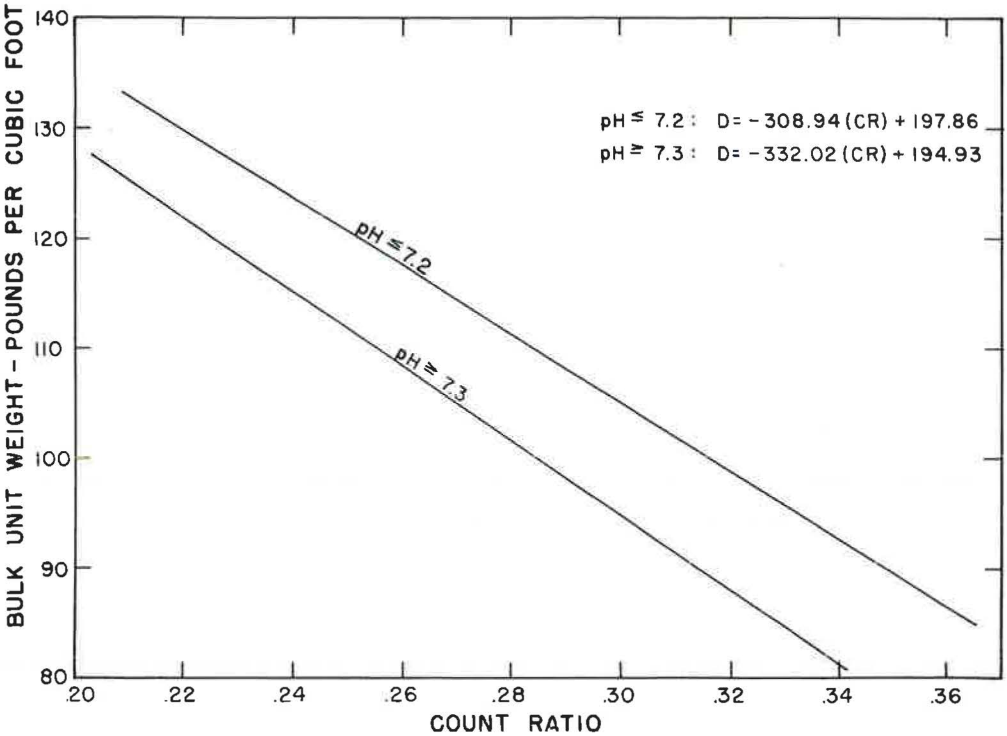

Figure 14. Density calibration curves for fine grained soils for instrument No. 1.

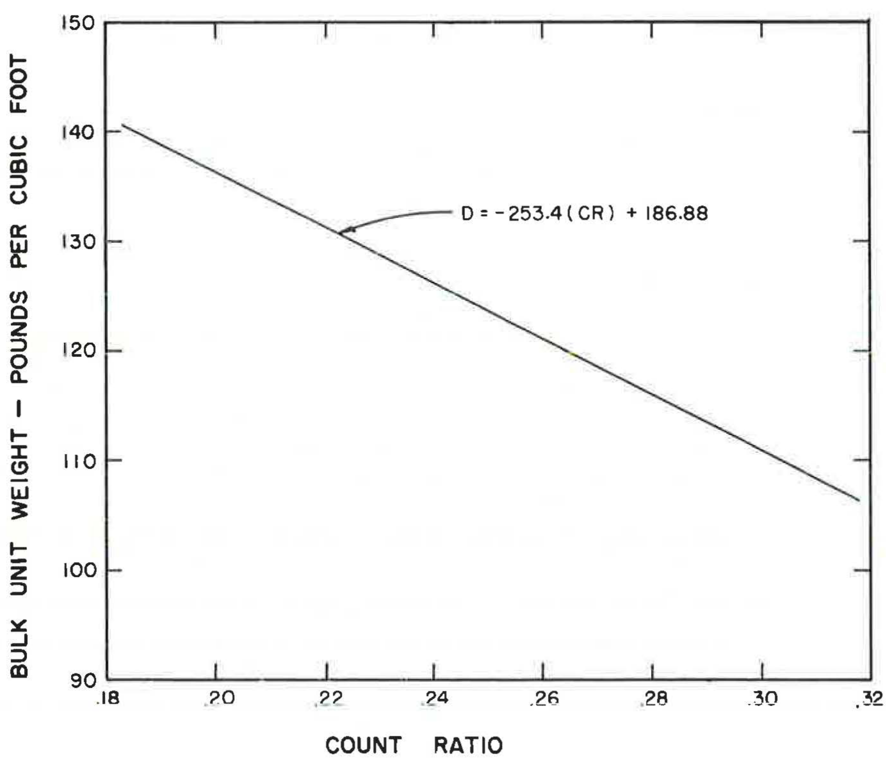

Figure 15. Density calibration curve for granular materials for instrument No. 1.

count ratio vs wet density as determined by the sand-cone test. The number of tests required to establish a calibration curve will vary from material to material and the final judgment will have to be made by the engineer.

IV. Summary—The guidelines presented should make it possible to adopt the nuclear gages for the routine control of field compaction. It is felt that as field data are collected, a further insight into the method of obtaining calibration curves may be gained. For the density gages, it is now felt that a single calibration curve cannot be valid for all soils. On the other hand, it would seem that a calibration curve for each soil would be impractical from the standpoint of the difficulty involved in gathering this amount of data. The best alternative at the present time appears to be in the adoption of a family of calibration curves based on soil pH. This can be developed after further field data are gathered and the results analyzed.

It appears that a single calibration curve for the moisture gage can be adopted for the materials commonly found in highway construction. As more field data are obtained, this concept can be further investigated. It is highly desirable that a detailed record be kept of all nuclear data obtained in the field. Specific importance should be placed on determining material composition and its effect on nuclear readings.

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Appendix

STATISTICAL DECISION PROCEDURE FOR CALIBRATION CURVE ACCEPTANCE

This Appendix deals with the development and suggested format to be used in conducting a statistical study of acceptance for a calibration curve to be employed with nuclear backscatter devices.

The exact procedure used in a study of this nature is a complex analysis, and perhaps beyond the level of present methods of statistical analysis. To provide a solution compatible with acceptable significance test methods and present knowledge of the distribution effects of the variables involved in nuclear backscatter devices, several simplifying assumptions have been made. The analysis is based on a significance test between a given calibration curve (laboratory or field developed) and the "true" calibration curve that the tested material inherently possesses.

The calibration curves were developed using a regression analysis. In all curves the independent variable was considered to be the nuclear count ratio reading and the dependent variable as either density or moisture. A condition required by the least squares analysis is that the error in the independent variable (count ratio) is small (i.e., a fixed value) compared to the variability of the dependent variable (density or moisture). The assumption of this condition was made for both density and moisture calibration curves.

It is recognized that the validity of this assumption can be questioned. The error associated with a count ratio of a moisture gage may be as large as the variability of moisture measurement by standard oven-drying techniques. The assumption might be more valid for the density calibration curve because the variability of sand-cone density determinations may be as large as ± 4.9 pcf (2).

Although the distribution of nuclear count readings is Poisson, the distribution of a count ratio reading is unknown as it is a ratio of Poission distributions. The situation is further complicated by the fact that as the number of 1-min tests used to determine an average nuclear count is increased, the distribution of the count may approach normality. Consequently, the distribution of a count ratio may range between a ratio of Poisson-distributed random variables to a ratio of normally distributed random variables.

Another aspect of the calibration curves that was investigated was the homogeneity of variance along the regression lines. This was done to check uniformity of variances over the entire range of data used to establish the regression lines. Cochran's test for homogeneity was used. Results for both density and moisture calibration curves did not reject the hypothesis of homogeneous variances for a level of significance (α) of 0.05.

A significance test can be used to test the hypothesis that the mean of a normal distribution has a specified value. If the actual density or moisture from a check test minus the predicted value obtained from the calibration curve is defined as the random variable and is normally distributed, then the optimum procedure for testing the hypothesis that the mean of this difference is equal to zero is based on the test statistic

$$t = \frac{\left[(\overline{X_A - X_P}) - u_0\right]\sqrt{N}}{S} \qquad (7)$$

where

t = test statistic,
X_A = actual density or moisture determined by check test,
X_P = predicted density or moisture determined from calibration curve,
$\overline{(X_A - X_P)}$ = average difference of N observations,
μ_0 = expected value of $\overline{(X_A - X_P)} = 0$,
N = number of check tests, and
S = standard deviation of N observations.

The variable $\overline{(X_A - X_P)}$ is assumed to be independently normally distributed over the entire count ratio range.

The decision to either accept or reject the calibration curves can be denoted by

$$\begin{aligned} H&: \overline{(X_A - X_P)} = 0 \\ A&: \overline{(X_A - X_P)} \neq 0 \end{aligned} \tag{8}$$

where (H) is the hypothesis that the predicted and true calibration curves are identical and (A) represents the alternative that they are not identical. Since it is possible for the true calibration curve to be either above or below the predicted calibration curve, a two-sided t-test is conducted. If the value taken on by the test statistic as a result of the check test falls in the rejection region, then the calibration curves cannot be adopted. Likewise, if the value is within the acceptance region, the calibration curves are accepted for field use.

The probability of rejecting the hypothesis when it is really true is the probability of the Type I error (α). In this case $\alpha = 0.05$. The probability of accepting the hypothesis when it is really false is called the Type II error (β); β is not known unless a specific alternative is given. This alternative is a judgment decision associated with accepting the calibration curve when in reality it should not be used.

Determination of the Required Number of Check Tests

The decision of the acceptable β error must be decided before the number of check tests required can be determined. The associated risk of the β error is a function of the true difference between means ($|\mu - \mu_0|$), the standard deviation (σ), and the number of tests required (check tests). Therefore, by selecting a value that corresponds to the difference in means (actual value minus predicted) a β error is then the probability of not detecting this difference when a sample size of (N) is used for a given α and σ. The difference in means selected for the density calibration curves of instrument No. 1 was 4 lb/cu ft: that is, when the difference between calibration curves is greater than 4 lb/cu ft, the given calibration curve is unsatisfactory. If this occurs, the probability of accepting the given calibration curve when it should be rejected is β. This difference for both moisture calibration curves was arbitrarily chosen to be 1.5 lb/cu ft of moisture. Figure 16 shows the probability of Type II errors associated with the number of check tests used for the various nuclear gages. The standard deviation (σ) of the random variable is unknown. Therefore an estimate of it must be made to determine the required number of check tests for a given β. The standard deviation (σ) was taken to be 3.0 lb/cu ft for the density gage and 1.0 lb/cu ft for the moisture gage. The decision was based on an examination of the calibration data.

As an example, it is desired to determine the number of check tests required for the instrument No. 1 density gage. The β error for determining a mean difference of 4 lb/cu ft between the given calibration curve and the true calibration curve for the soil in question should be 0.05. From Figure 16, the required number of check tests is found to be 10. If conditions at the job site were such that only 6 check tests were conducted, the β error for the decision would be approximately 0.25. That is, the probability of accepting the hypothesis that the calibration curve is valid when it is really false is 0.25 if only these 6 tests were used.

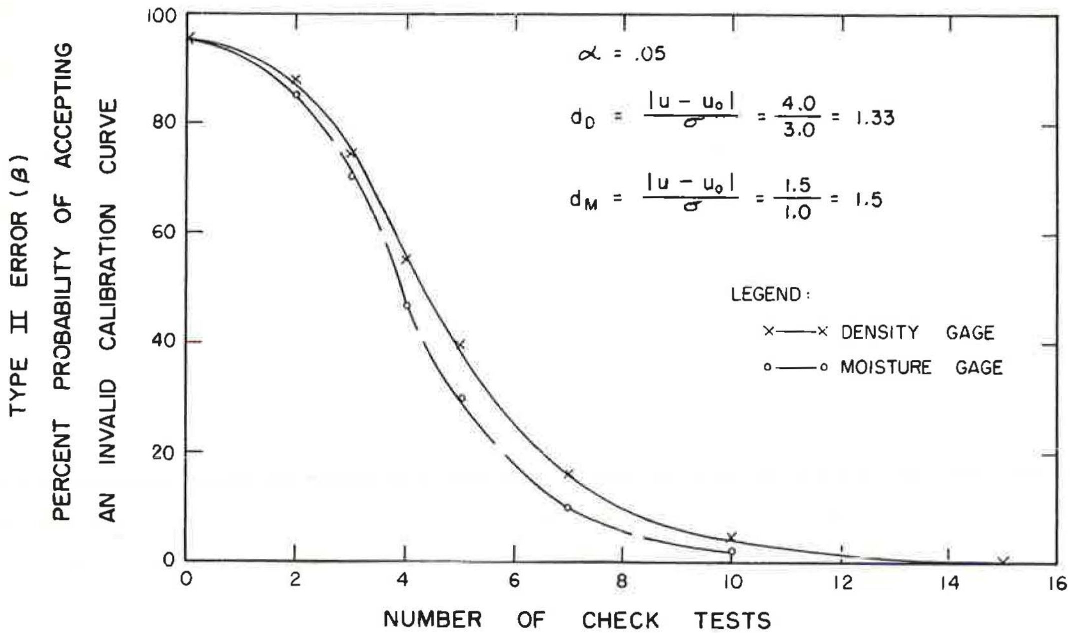

Figure 16. Type II error associated with number of check tests for different nuclear gages.

Significance Test Procedure for Calibration Curve Acceptance

It has been previously stated that the t-test is used to decide whether a given calibration curve will be accepted or rejected for use with a nuclear gage. The discussion of the t-test can be found in most statistics books. Therefore, only a suggested format for arriving at a decision based upon computation of the t statistic is given:

1. Determine the required number (N) of check tests necessary based on acceptable probability of Type II error (β) (Fig. 16). It is suggested that a β error of 0.05 be adopted.
2. Conduct the (N) check tests as previously described.
3. Determine the predicted density or moisture value by the appropriate calibration curves for the count ratio found from the check test.
4. Calculate the difference in density or moisture determined from the check test and the predicted value found in step 3 for each check test (X_A - X_P).

TABLE 3

VALUES OF t′ FOR VARIOUS NUMBER OF CHECK TESTS AT 0.05 LEVEL OF SIGNIFICANCE

$\left[t' = t_{(\alpha/2,\ \nu)} \right]$ where $\nu = N - 1$

N	t′	N	t′	N	t′
2	12.71	9	2.306	16	2.131
3	4.303	10	2.262	17	2.120
4	3.182	11	2.228	18	2.110
5	2.776	12	2.201	19	2.101
6	2.571	13	2.179	20	2.093
7	2.447	14	2.160	21	2.086
8	2.365	15	2.145	22	2.080

5. Compute $(\overline{X_A - X_P})$ based on (N) observations.
6. Compute the standard deviation of the N observations.
7. Compute the value of the test statistic t.
8. Determine from Table 3 the test statistic t' based on N observations for $\alpha = 0.05$.
9. If $(-t' \le t \le t')$ accept the calibration curve field control.
10. It $(t > t')$ or $(t < -t')$, recalibration for the particular soil in question must be accomplished.

TABLE 4

Suggested Worksheet for Significance Test Computation

Instrument No.(Name):____________ (Density)(Moisture) Calibration Curve No.:__________

Project No.:____________________

(Selected β Error:________________

Required No. of check tests:________

No.	X_A (1)	CR_A	X_P (2)	$X_A - X_P$ (3)	$(X_A - X_P)^2$ (4)
1					
2					
N					
				$\sum(X_A-X_P)$ (5)	$\sum(X_A-X_P)^2$ (6)

Where: X_A = Check test (density) (moisture)

CR_A = Check test Count Ratio

X_P = Predicted (Density)(Moisture)

X_A-X_P = Difference of (Density)(Moisture)

$(X_A-X_P)^2$ = Square of difference

A. Calculate average difference

$$(\overline{X_A-X_P}) = \frac{\sum(X_A-X_P)}{N} = \frac{(5)}{N} *$$

B. Compute Standard deviation (S)

$$S^2 = \frac{\sum(X_A - X_P)^2 - \frac{\left[\sum(X_A - X_P)\right]^2}{N}}{N-1}$$

$$S^2 = \frac{(6) - \frac{(5)^2}{N}}{N-1}$$

$$S = \sqrt{S^2}$$

C. Compute test statistic (t)

$$t = \frac{(\overline{X_A-X_P})\sqrt{N}}{S}$$

$$t = \frac{A\sqrt{N}}{B} = \underline{\qquad\qquad}$$

D. Determine t' from Table 4 based on N observations

t' = ______________

E. Use calibration curve if

$-t' \le t \le +t'$ or

$-D \le C \le +D$

F. Do not use calibration curve otherwise

* Numbers in ( ) reference to column numbers.

Compaction Control of Granular Base Course Materials by Use of Nuclear Devices And a Control Strip Technique

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In an attempt to overcome some of the problems encountered in the compaction control of granular base materials through conventional methods, Virginia has recently developed a new approach. A control strip is constructed by the contractor, a density standard is established through nuclear moisture-density testing, and this standard is used as the basis for controlling the compaction of other sections built with like material. The method has proven to be very satisfactory on three projects, and will be used on eight more that are now ready for advertisement.

•IN MOST conventional methods of compaction control of granular base course materials, some weak points exist. These can be summarized briefly as follows:

1. Tests are time-consuming. The conventional method of digging a test hole, determining the weight and moisture content of the material removed, and the volume of the hole is tedious and time-consuming. This can impede construction.

2. Maximum density must be determined in the laboratory. The determination of the maximum density of the fine portion of a base course material is relatively simple. However, when the material contains an appreciable amount of coarse fraction, a correction is necessary. No single method for determining the correction factor is widely accepted.

Laboratory compaction tests for the total sample are available; however, they are not widely used because inclusion of the coarse fraction necessitates the use of relatively large molds and introduces such factors as degradation and wall friction.

The values obtained by both the use of correction factors and tests on the total sample have been questioned in some cases because they have not been obtainable in the field regardless of the amount of compaction effort.

3. Methods give a poor estimate for acceptance or rejection. Since conventional tests are time-consuming, one value is taken to represent a large volume of material. This one value provides a poor estimate on which to base acceptance or rejection, because high variability might exist.

It is not the intention of the authors to condemn conventional tests on the basis of the weak points summarized, but rather to note that the method offered in this paper can overcome these inadequacies because of the following features:

1. Nuclear tests can be made quickly and easily;
2. A field control strip provides a practically achievable density; and
3. The speed of nuclear testing permits determinations to be made for each section of material, which provides a sound statistical basis for decision making.

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

GENERAL PROCEDURE

The Control Strip Technique

The control strip technique is by no means a new concept. It has been used by some states, notably Ohio, for many years. In general, the technique involves the construction of a control strip of the material at the job site. This is achieved by selecting an area on a firm subgrade or subbase and rolling it in increments of compactive effort with equipment of a specified minimum weight, and with the material at optimum moisture content as determined in the laboratory and corrected for the coarse fraction. To obtain a roller pattern, density tests are performed after each rolling until no further increase in density is detected.

The completed control strip becomes a part of the construction and the rest of the project is controlled in larger "test sections" in each of which the density must be a certain percentage of that of the control strip. In these test sections, however, neither the moisture content nor compaction equipment is controlled by the enforcing agency. Failure to achieve the required density within a section means additional rolling and retesting. A new control strip is required when a change in material is detected. The whole technique is predicated on the fact that the gradation of the material remains within specified limits.

Use of Nuclear Equipment

In both the construction of the control strip and the testing of the test sections, a number of determinations sufficient for providing the desired accuracy are required. This means that several density tests must be made. Conventional density tests are too time-consuming and therefore not practical for this purpose. Nuclear methods, on the other hand, being quite rapid, can be used successfully (1-min moisture and 1-min density readings constitute a test in this procedure). Any sufficiently sensitive calibration curve can be used since any effects from chemical composition, surface texture, etc., encountered in the test section have been encountered in the control strip. However, since this method is nondestructive, if "crusting" occurs, that is, if there is a greater density on top than on the bottom, it can be passed undetected.

Specific Procedure

Rolling Pattern—The roller pattern is obtained on the control strip, a 300-ft section of one-lane roadway. Figure 1 shows a typical roller pattern after each pass of a vibratory roller. Each point is the average of three tests taken on the control strip. This figure shows that the maximum attainable dry density was about 139 pcf and that it occurred after eight passes with the roller.

Control Strip—In order to obtain a very good estimate of the dry density of the control strip after the maximum density has been reached, ten random moisture and density tests are run. This number of tests provides a very good indication of the dry density of the material, and a percentage of this figure is used to determine compaction compliance on the remaninder of the project.

Test Section—Each 2000 ft of one-lane roadway is then designated as a test section. Each section is tested randomly at five locations. From a statistical analysis, it has been found that the average of these five tests should be at least 98 percent of the average obtained on the control strip, and each individual test value should be at least 95 percent of the average control strip density.

EXPERIENCE GAINED

Experimental Project

During the summer of 1964 the compaction of the aggregate base course of a project on Route 6 in central Virginia was constructed using the control strip technique with the nuclear equipment and then the level of compaction checked by conventional procedures.

Experience on this project indicated that the control strip technique way as sensitive as the conventional procedures. Furthermore, the conventional density tests run indi-

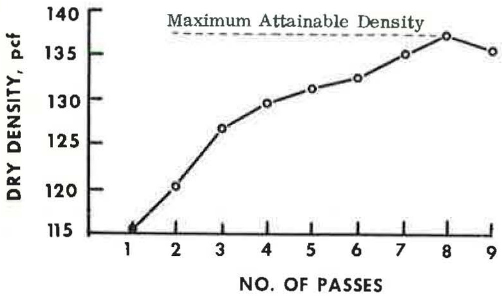


Figure 1. Roller pattern, vibrations roller.

cated that the compaction level achieved was equal to or above that desired. This initial work indicated that both the contractor and testing personnel were satisfied with the method.

Additional Projects

Encouraged by the results of the work done on Route 6, three more projects were let using the control strip technique for compaction control. Two of these projects have been successfully completed and one is well under way. No major difficulties have been encountered thus far.

As an indication of the type of data obtained on these projects, Appendix A, which includes typical data for roller pattern, control strip and test section, was prepared.

Current Status

As experience is gained with this technique and funds become available for the purchase of additional nuclear gages, more projects are being advertised using this technique. This past fall eight more projects, one in each construction district of the Virginia Department of Highways, were let to contract. The special provision governing the use of this technique is shown in Appendix B.

SUMMARY

The method described in this paper has several advantages and some disadvantages as compared to conventional test methods.

Advantages

1. The use of nuclear methods results in a better estimate of the variability because the data lend themselves to statistical analysis;
2. No laboratory test for density is required;
3. No correction for gradation of the material is required;
4. Testing is physically easier and more rapid;
5. Any calibration curve can be used with nuclear devices as long as the sensitivity of the curve is adequate; and
6. Psychological advantages exist for contractor and testing personnel since for the project the contractor is asked only to achieve a certain percentage of the density he has achieved in the control strip.

Disadvantages

1. Since the method is nondestructive, the distribution of density throughout the base course cannot be detected. If crusting occurs, it can be passed undetected. (There has been no indication that crusting is actually a problem and this condition should not occur when proper equipment is used.)
2. The cost of nuclear equipment is much higher than that of conventional equipment. If, however, one realizes the higher level quality control achieved with the same amount of time, then the initial price difference can be tolerated.

In conclusion, it can be said that, based on the experience gained within the last few years, the use of the control strip technique with the nuclear devices can be very successful and is recommended. It is apparent to the authors that this system has certain disadvantages, but these are far outweighed by its advantages.

Appendix A

ROLLER PATTERN

Date 7-18-66 Project 0029-039-101 6501

Section No. Roller pattern #3 Sta. 826 + 50 to Sta. 829 + 50

Type Material SBM-1 Width 26' Remarks: 6" deep

Standard Count

Density	Moisture
60982	9300
60967	9362
61497	9608
61549	9550

Total 244995 Average 61249 Total 37820 Average 9455

Test 1 — after 3 passes with vibratory roller

Station	Density	Moisture
826 + 75	38270	4094
827 + 50	39295	4137
828 + 25	38550	4009
Total	38705	4080
C.R.	.632	.431

Test 2 — after 6 passes with vibratory roller

Station	Density	Moisture
826 + 75	35077	4579
827 + 50	37190	4280
828 + 25	35871	4368
Total	36046	4409
C.R.	.589	466

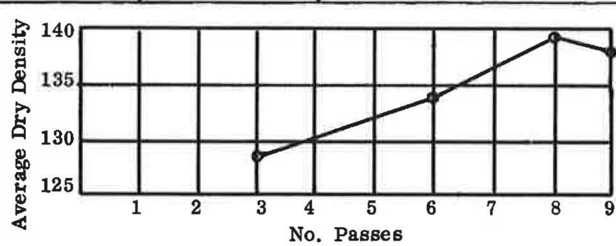
Test 3 — after 8 passes with vibratory roller

Station	Density	Moisture
826 + 75	36602	4571
827 + 50	36212	4246
828 + 25	34826	4383
Total	34880	4400
C.R.	.569	.465

Test 4 — after 9 passes with vibratory roller

Station	Density	Moisture
826 + 75	35423	4406
827 + 50	36494	4227
828 + 25	33770	4500
Total	35229	4378
C.R.	.587	.463

Test	Wet Density —	Moisture	= Average Dry Density
1	137.3	8.0	129.3
2	144.0	9.2	134.8
3	146.8	9.1	137.7
4	144.5	8.6	135.9



CONTROL STRIP DENSITY

Date 7-26-66 Project 0029-039-101-6501

Type Material SBM-1

Sta. 826 + 50 to Sta. 829 + 50 Width 26'

Depth 6"

Standard Count

	Density				Moisture	
	61240				9459	
	61207				9505	
	61310				9592	
	61105				9591	
Total	244862	Mean 61216		Total	38147	Mean 9536

	Test	Station	Density	Moisture
	1	826 + 50	33039	4765
	2	826 + 65	36687	4747
	3	826 + 80	32133	4603
	4	826 + 95	34874	4604
	5	827 + 10	35770	4678
	6	827 + 25	34275	4532
	7	827 + 40	34436	4595
	8	827 + 55	33775	4724
	9	827 + 70	34637	4666
	10	827 + 85	34710	4860
Total			344336	46774
Mean			34433	4677
C.R.			.562	.490

Wet Density 147.5 — Moisture 11.0 = Dry Density 136.5

Dry Density Requirement

(.98)	(136.5)	= 133.8	= Mean Density Requirement
(.95)	(136.5)	= 129.7	= Individual Density Requirement

TEST SECTION DENSITY

Date 7-27-66

Project 0029-039-101 6501

Section No. 12

Sta. 796 + 50 to Sta. 814 + 50

Type Material SBM-1

Width 13' Rt.

Remarks 6" Deep

Standard Count

	Density			Moisture	
	61099			9538	
	61046			9458	
	61027			9540	
	61227			9547	
Total	244399	Mean 61100	Total	38083	Mean 9521

Test	Station	Density	Moisture
1	796 + 50	35118	4709
	C.R.	.575	.495
2	798 + 50	35158	4444
	C.R.	.575	.467
3	800 + 50	35196	4612
	C.R.	.576	.484
4	802 + 50	33448	4701
	C.R.	.547	.494
5	804 + 50	36139	4841
	C.R.	.591	.509

Sample	Wet Density −	Moisture =	Dry Density	Requirement	Passing
1	145.8	11.2	134.6	129.7	✓
2	146.0	10.2	135.8	129.7	✓
3	146.0	10.8	135.2	129.7	✓
4	150.0	11.2	138.8	129.7	✓
5	143.5	11.7	131.8	129.7	✓
Mean			135.2	133.8	✓

Appendix B

VIRGINIA DEPARTMENT OF HIGHWAYS
SPECIAL PROVISIONS FOR
NUCLEAR FIELD DENSITY TESTING OF
AGGREGATE BASE AND SURFACE COURSES

February 23, 1965
Rev. 10-19-66

Section 308 of the 1966 edition of the Road and Bridge Specifications is amended in this contract to require the construction of density control strips for the purpose of using the nuclear field density testing device. The revisions are as follows:

At the beginning of the work the Contractor shall build a control strip of the material on an approved and stable subgrade for the purpose of the Engineer's determining density requirements for the project. This control strip will be at least 400 square yards in area and of the same material and depth to be used in the remainder of the work. Compaction will be carried out with conventional rollers approved by the Engineer until no appreciable increase in density is accomplished or until in the opinion of the Engineer no appreciable increase in density will be obtained by additional rolling. Upon completion of the rolling, the density of the strip will be determined by use of a portable nuclear test device.

The compaction of the remainder of the aggregate base course material shall be governed by the density of the control strip. The material shall be tested by sections of approximately 2800 square yards each. The mean density of 5 randomly selected sites from the test section shall be at least 98 percent of the mean density of 10 tests taken from the approved control strip. Placing, compacting and individual testing may be done in subsections of approximately 280 square yards each. When the mean of the test section is less than 98 percent of the control strip mean, the Contractor may be required to rework the entire section. Also, each individual test value shall be at least 95 percent of the mean value of the control strip. When an individual test value is less than 95 percent of the control strip mean, the

Contractor shall be required to rework the area represented by that test.

Each test section shall be tested for thickness and any deficiency outside the allowable tolerance shall be corrected by scarifying, placing additional material, remixing, reshaping and recompacting to the specified density.

A new control strip may be requested when:

(1) A change in the source of the material is made, or

(2) a change in the material from the same source is observed, or

(3) ten (10) test sections have been approved without the construction of additional control strips.

Note: The Contractor's attention is directed to the fact that the method for determining density and the requirements for density as described in Section 308.05 have been replaced by the method of determination and requirements for density stated hereinabove.

Practical Applications of the Area Concept to Compaction Control Using Nuclear Gages

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Respectively, Senior Materials and Research Engineer, and Assistant Materials and Research Engineer, California Division of Highways, Sacramento

The rate of placement of earthwork in highway construction has greatly expanded since World War II; however, the acceptance or rejection of this earthwork compaction has been based on prewar methods that are geared to lower production rates. The California Division of Highways has been developing a new test method for accepting or rejecting earthwork compaction. This method has three important facets: (a) a modified statistical approach, (b) the use of nuclear soil gages, and (c) an area concept.

The statistical approach consists of obtaining several in-place densities of the compacted earthwork in an area to be tested. The acceptance or rejection is based on the average relative compaction and the percentage falling below the required relative compaction value. The test sites are somewhat randomly selected in an area ready for testing. The area is passed or failed according to the test results. The density of the compacted material is determined by use of nuclear soil moisture and density gages. This new test method was used experimentally on a project during 1964 and the results were satisfactory. After some modification of the test procedure it was used on several projects in the 1965 and 1966 construction seasons. This compaction control concept was successful from both the state and contractors' points of view.

•THE ART of controlling compaction of embankments in California has varied only slightly since its inception in 1929 (1); however, the rate of placing embankments has increased about tenfold. The increased production has made compaction control difficult using previously acceptable methods. In an attempt to reduce the time required to determine the percent relative compaction, the California Division of Highways introduced the "wet method" (2) in 1954, which largely eliminated the necessity for oven-drying moisture samples. The use of nuclear surface gages was investigated starting in 1959 to determine the in-place moisture and density of compacted earthwork (3). As a portion of the field studies with the nuclear gages, statistical studies of the variation in density of compacted earthwork were conducted. In 1964, a study of the density variation within accepted embankments was conducted on three projects (4). As a result of these studies a new test method for compaction control, using a modified statistical approach and a nuclear surface gage, was tried on 11 construction projects throughout California in 1965 and 1966 (5).

Paper sponsored by Committee on Construction Practices—Earthwork and presented at the 46th Annual Meeting.

TEST METHOD

In 1964, a nuclear gage was used to control earthwork compaction on a project in the north coastal area of California Highway District 01. The test method specified that multiple testing was to be used; that is, several tests were to be made with the nuclear gage at each of several locations in the area. The individual nuclear test just below the average value of all the tests was used to compute the relative compaction. The acceptance or rejection of an area was thus dependent on the average of several nuclear in-place density tests. On this project the number of tests in a given area varied from 2 to 15. The multiple testing concept was intended to compensate for the variation in the results indicated by previous work in California (3) when nuclear gages were used. In analyzing the data from this preliminary project (6), it was noted that the average value did not take into account the spread or range of the in-place densities. It became apparent that a statistical approach was desirable.

The accepted embankments, with a 90 percent relative compaction requirement, had a range of relative compaction from 80 percent to 106 percent, an average of 95.2 percent with a standard deviation of 4.2 percent. While the majority of the individual tests and all of the average values from the passing areas were at or above the minimum 90 percent relative compaction specification for the embankments, it can be seen from Figure 1 that there was a small group of substandard values scattered through

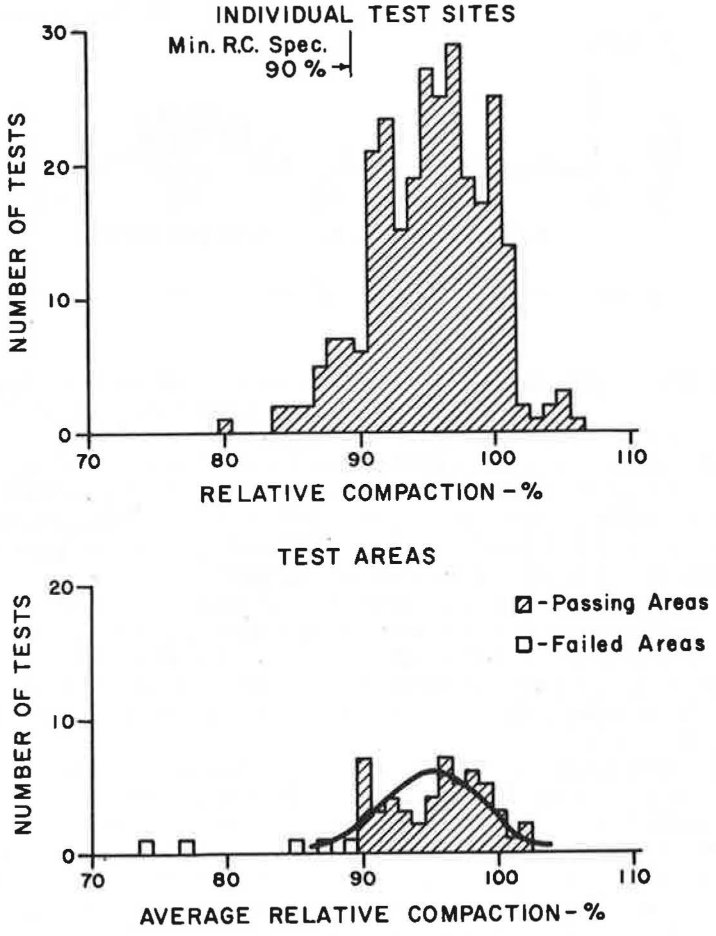

Figure 1. Frequency distribution, pilot project—embankment.

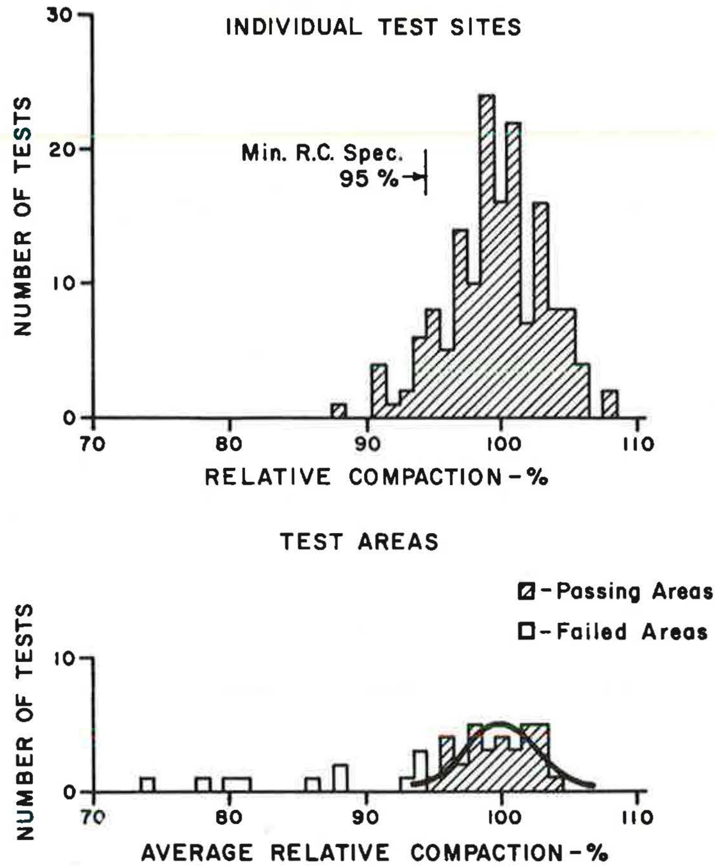

Figure 2. Frequency distribution, pilot project—processed material.

the areas. These substandard tests represent about 9 percent of the total tests from the passing areas. This compares well with the AASHO Road Test (7), where 8.8 percent of the tests fell below the specification limit.

Test results on the structure backfill, aggregate subbase, and aggregate base showed a pattern similar to the embankment tests (Fig. 2). The passing areas ranged from 88 percent to 108 percent, an average of 99.8 percent relative compaction with a standard deviation of 3.4 percent. About 8 percent of the tests from the passing areas fell below the 95 percent minimum specification.

The three projects reported by Jorgensen and Watkins (4) indicated that the range in relative compaction was 87 to 98, 85 to 97 and 80 to 103 percent with averages of 92.9, 90.5 and 93.6 percent, and standard deviations of 2.4, 3.1 and 5.5 percent. These tests were all from areas accepted by the present sand volume test method. This study confirmed the findings on the first project where a nuclear gage was used for construction control.

The distribution curves for the embankment and processed material averages are shown in the lower halves of Figures 1 and 2. It is to be expected that the passing area will only extend from the relative compaction specification limit upward, since the failed areas are normally reworked and retested until they become passing areas. However, this does not present an entirely true representation of the probable final state of compaction. Only a very small portion of the total volume of the soil was tested, and some areas not tested would be expected to be below the relative compaction specification limit.

It was felt that there were two advisable ways of modifying the multiple testing procedure which would tend to minimize the chance of including substandard compaction in the final product. First, there should be some limitation placed on the percentage of failing tests allowed in an area having a passing average and still have the area acceptable. Second, there should be some measure of control on the spacing and minimum number of individual test sites within an area.

The first modification was determined by studying all the available data on the in-place density variation in acceptable compacted fills. Jorgensen and Watkins (4) reported that the percentage of tests below the specified minimum relative compaction varied from 8.5 to 43 percent. A review of AASHO test road compaction data indicated that 8.8 percent of the tests were below the specified relative compaction, in acceptable areas. In 1953, Davis (8) reported that 10 to 25 percent of the tests in acceptable areas in dam construction were below the specified relative compaction. The ideal situation would be where the type of material determined the percentage of failing tests, since the percentage of failing tests should be lower with more uniform material. However, this would be difficult to determine in advance of construction.

These variations of in-place density represent the variation within the compacted soil mass and the variation in the sand volume test procedure. Previous work in California indicated that the nuclear test method had a larger variation than the sand volume when used to determine in-place density (3). Considering all these variations, it was decided that in order to obtain the same compaction as at present, not more than one-third of the individual tests in any area should be below the specified minimum relative compaction.

The second modification was decided on the basis of statistics. Five or six tests were required for a 95 percent confidence level on acceptance or rejection for an estimated average area (see Appendix, also Ref. 9). With the one-third failing requirement, it was decided to use six individual tests per area. For the location of the tests, standard control practices in industry and the recommendations of Miller-Warden Associates (10) were studied. It was decided to use a basic unit as an area to be accepted or rejected. This area is then divided into two or more subareas of approximately equal size. Two or more nuclear tests are taken in each subarea. Locations of individual test sites were selected at random. This allows flexibility of action by the resident engineer in controlling compaction, and still retains the basic elements of statistical concepts. This new testing concept was called the area concept and was worded in the test method somewhat as follows:

NUMBER AND LOCATION OF NUCLEAR TESTS

The nuclear test will utilize the area concept; that is, a series of tests will determine whether to accept or reject an entire area. Perform six or more nuclear tests in each area. The engineer shall determine the area based on uniformity of factors affecting nuclear testing.

Divide the area into two or more subareas of approximately equal size. Perform two or more nuclear tests upon each subarea with the locations of the nuclear tests being of a random nature. (For special cases one subarea may be tested with three nuclear tests and considered an area.) Determine the moisture and density of the soil by the nuclear tests as described elsewhere in the procedure.

Average these six or more tests and perform the maximum density test on the soil obtained from the location of the nuclear test which has a value just below the average value. Determine the maximum density as specified in Test Method No. Calif. 312 for Classes A and B CTB and Test Method No. Calif. 216 for all other treated and untreated soils and aggregates.

Care must be taken that the same soil type exists over the given area. This is so that the one maximum density test is consistent with the nuclear tests.

Using the maximum density test, calculate the percent relative compaction for each nuclear test. The average of all of the nuclear determined relative compaction tests must be above the required compaction value. No more than one-third of the individual tests may be below the required compaction value. If the average of all tests in one subarea fail to meet the required compaction value, this subarea may be failed even though the other subareas may be passed. Thus, either subareas or areas may be passed or failed.

When sufficient maximum density tests have been obtained, a value may be established for a soil type and only perform check maximum densities on that soil type at least weekly.

Discussion of Test Method

The test method: (a) must be reasonably rapid in obtaining results, (b) must allow the resident engineer to use his discretion and engineering judgment as to the application of the test method, and (c) must be simple and clear in operation so that field personnel need not spend an excessive amount of time interpreting the results.

Individual tests can be obtained in 5 to 15 minutes by using nuclear surface gages. This means that a complete set of six tests could be made in about one hour. The previous sand volume test method (11) required that a standard compaction test be conducted at each test site, but in the new test method (12) one standard compaction test is conducted for an area. By averaging several standard compaction tests, a complete area could be tested in about an hour. In general, the success of this test method depends on the uniformity of the soil type and the use of the nuclear surface gages. The pilot project in District 01 had indicated that there was every reason to expect successful use of the nuclear gages.

It was strongly felt that discretion and engineering judgment must be retained by the resident engineer. The test method was prepared as a guide to the resident engineer and was designed to be flexible and adaptable to changing job conditions. Consideration was given to limiting the size of an area to be tested. In reviewing the data from the District 01 pilot project, it was found that up to 3 miles of subbase was tested using 15 individual tests. The only limitation on the size of the area was to reduce the number of individual tests to 3 where limited areas such as pipe pads and structural backfill were being tested.

The test method should avoid technical and complicated procedures. In normal statistical work a table of random numbers would be used; however, this is time-consuming and requires additional training. To avoid or minimize operator bias, the system of subareas was used. This distributed the tests over the entire area. The selection of at least 2 individual test sites in a subarea would allow for bias in the individual test site selection. However, the need to randomize the testing was stressed, and the potential bias was considered a slight risk. The acceptance or rejection must be in clear and concise terms. The statistical procedures used in industry could not be applied to earthwork due to the lack of control over the original material. To provide a quick and simple method of determining acceptance or rejection, two simple guides were used: (a) the average value and (b) the permissible percent of the total tests below the accepted minimum. Consideration also was given to using an absolute minimum; that is, where any one individual test was below this relative compaction it would result in rejection of an area. However, it was felt that this was an unnecessary addition to the acceptance or rejection criteria.

Field Use

After reviewing California's work with nuclear gages and related compaction studies, it was decided to use the nuclear gage in an experimental program. Five transmission gages and five backscatter gages were employed. The new area concept was specified as the method of accepting or rejecting the compacted earthwork.

The research program was arranged so that nuclear gages were used on 11 projects in 10 highway districts during the 1965 and 1966 construction seasons, with a few projects to be completed in 1967. This provided a broad range of various soil types, terrain, climatic conditions and construction operations which represented a cross section of typical situations encountered in California. Quantities of embankment and structural section material varied from about $\frac{1}{4}$ to $15\frac{1}{2}$ million cubic yards per project. Thus, the nuclear gages were required to check compaction compliance on over 45 million cubic yards of material.

Ten nuclear surface soil gages were purchased from four manufacturers and the previously purchased gages were used as spare gages. One nuclear gage was used on two projects. Of the four makes, two were backscatter type and two were transmission type. Thus, a comparison of the backscatter and transmission type gages was available.

A one-week training course was conducted for the resident engineer, a progress tester, and at least two technicians for each project. The course covered basic nuclear physics, health safety, gage operation and test method concepts.

The resident engineers were responsible for the application of the new test method, application of the nuclear gages to the test method, maintenance of weekly health safety records, and consideration of nuclear source storage and transport. Each operator and the resident engineer was equipped with film badges and dosimeters to monitor radiation exposure.

At the present time, six projects are completed and the remainder will be completed in 1967 or later. Sufficient information was obtained by the end of the 1966 construction season to decide on the future use of nuclear gages as well as the area concept. At a meeting in January 1966, resident engineers discussed the technical aspects of the new test method and how it was performing in the field. In July 1966, district meetings were held with representatives of the Materials and Research and Headquarters Construction Departments and district field and supervisory personnel to discuss the general administrative aspects of the test method, and functions the district would be required to assume in the use of nuclear sources. In the fall of 1966, after executive level conferences, the new test method (No. Calif. 231) was adopted by the California Division of Highways (5).

Problems and Solutions

Calibration of Gages—The test method originally required the field calibration of the gages by comparison with sand volume tests. This resulted in considerable difficulty, regarding two items: (a) frequent recalibration of backscatter gages with changes of soil types, and (b) the use of nuclear gages to test soils where it was difficult or impossible to perform sand volume tests.

Several calibration curves were required on each project when using backscatter type gages. On one project nine calibration curves were used. However, with the transmission gage, one calibration curve was adequate for all of the soils, which is in agreement with previous work in California (3, 13). The problem was solved by specifying the use of the transmission type gage, which was calibrated using standard blocks in a central laboratory. Consequently, if calibration needs checking in the field, it may be done with either a large mold or by sand volume comparisons.

When it was impractical to obtain sand volume tests, the soil was compacted in an 18 by 18 by 12-in. mold. In this manner gages were calibrated for soils on which sand volume tests could not be obtained.

Site Preparation—It had been anticipated that site preparation for the individual nuclear tests would be one of the major problems (3). The complaints generally concerned two conditions: (a) a hard and somewhat clayey soil, and (b) a rocky soil. With the hard soil it was time consuming to cut a plane surface by hand, sometimes requiring $\frac{1}{2}$ hour or more per test site. The solution to this problem varied considerably. On one project it was found that a motor grader would prepare a satisfactory site, on another a scraper was found to work well. The primary problem with rocky soils was the depressions caused by rocks that were removed from the soil. This was overcome by compacting native fines by hand into the depressions. There was no general solution obtained to this problem; however, the time required for site preparation was reduced to a reasonable amount by various means. The site preparation procedure will thus vary from job to job depending on soil conditions; the test method must not be restrictive in this respect.

TABLE 1

NUCLEAR GAGE MALFUNCTIONS
MARCH 1, 1965 TO JULY 1, 1966

Cause	Occurrences (no.)	Working Days Downtime
Scaler	28	184
Probe	12	103
Cable and/or connections	18	43
Power supply	4	5
Binding of transmission rod	5	5
Total	67	340

Site preparation was a problem on all projects using the backscatter gages, but only a problem on one project using the transmission gage. (The site preparation was considered adequate when two readings obtained by rotating the nuclear gage 180 deg checked each other within about 3 lb/ft^3.) It had been anticipated that the greatest problem connected with the transmission gage would be due to the necessity of drilling a hole in the soil. However, this was not a problem on any project using the transmission gage. There were two methods used to make the hole in the soil—a driven pin and a power drill. Both methods were used about equally on the various projects using transmission gages.

Maintenance—Equipment maintenance was a major difficulty that developed in the field use of the nuclear gages. It was anticipated that some downtime would occur, so 12 gages were on hand to be used on 10 projects, leaving two spare gages. Two spare gages proved inadequate, and about one-third of the time there were no spare gages available. Experience showed that a spare gage should be available for every three nuclear gages used in the field, and they should be of the same make.

Downtime for individual gages varied from less than one day to one month. The number of downtime occurrences and the total times are given in Table 1. Because the cable was frequently the cause of a malfunction, a spare cable was obtained for each make of gage. The binding of the transmission rod only occurred on one make of gage, where the nuclear source was placed underground. The operators then had to handle the unshielded source and this had an adverse psychological effect. This downtime was overcome by weekly cleaning of the transmission rod and guide; however, the psychological effect remained.

The use of backup gages of the backscatter type was not successful, mainly due to the need to calibrate the gage to the soil type. This generally resulted in about a two-day or longer delay in getting the gage in operation. After several occasions where recalibration was required for use of the backup gage, the resident engineers using backscatter gages would refuse to use backup gages. This became a serious matter when a delay of several weeks occurred for repairs.

A backup gage problem did not occur on projects where transmission gages were used. On receiving the backup gage, immediate resumption of testing occurred because the predetermined calibration was of sufficient accuracy.

Backscatter-Transmission Comparison—One objective of the research program was to compare the backscatter and transmission gages in actual field operations. The principal disadvantages of the backscatter gages were the need to calibrate for each soil type, and their sensitivity to seating on the soil surface. Both of these problems have been discussed. With one transmission gage some difficulty was encountered in aligning the gage over the hole. The other transmission gage had an attachment so that the hole could be seen and the rod easily aligned over the hole. As the result of this research program a specification has been prepared recommending a transmission gage with the detector tube placed underground.

General Comments—The nuclear gages had many advantages over the sand volume test. One major advantage was the ability to test rocky soils that previously could not be tested using the sand volume technique. On project after project soils were tested that previously had been accepted on the basis of inspection. It was estimated that about 20 percent more rocky type material could be tested with the nuclear gage than with the sand volume test. On only two projects was material encountered that was untestable with the nuclear gage, and this was because a plane surface could not be obtained to seat the gage on.

Some time was saved on the individual test sites when the nuclear gage was used. With the sand volume test, it generally required ¼ to ½ hour per test, and with the six nuclear test sites it generally required ½ to 1½ hours. However, these times do not reflect the whole picture. Where dry densities were required, the sand volume test needed additional time to take a sample to the project laboratory and dry it. This was where the real time was saved on many projects. The ability of the nuclear gage to give an answer in the field without further work was a decided advantage.

An important item from the contractor's viewpoint was that he was not required to stop the equipment during the nuclear test. With the sand volume test all equipment on the fill would have to stop while the sand was being poured in the hole. It was quite a sight to see the heavy earthmoving equipment operating at full speed in the vicinity of the nuclear gage.

The decision whether to use the power supply provided with the nuclear gage or to use the vehicle battery for power was left up to the resident engineer. On three projects the resident engineer used the vehicle batteries to operate the nuclear gages. There were no downtimes due to failure in the power supply on these three projects, whereas on the other projects there were significant downtimes due to failure of the power supply provided with the gage. For this reason the specifications for the new gages require the use of the vehicle battery as the power source.

EVALUATION

Performance of Area Concept

At the start of this research program there was considerable concern about the acceptance of the statistical concept. In the training classes there was substantial reluctance on the part of the trainees to accept the statistical concepts; however, the

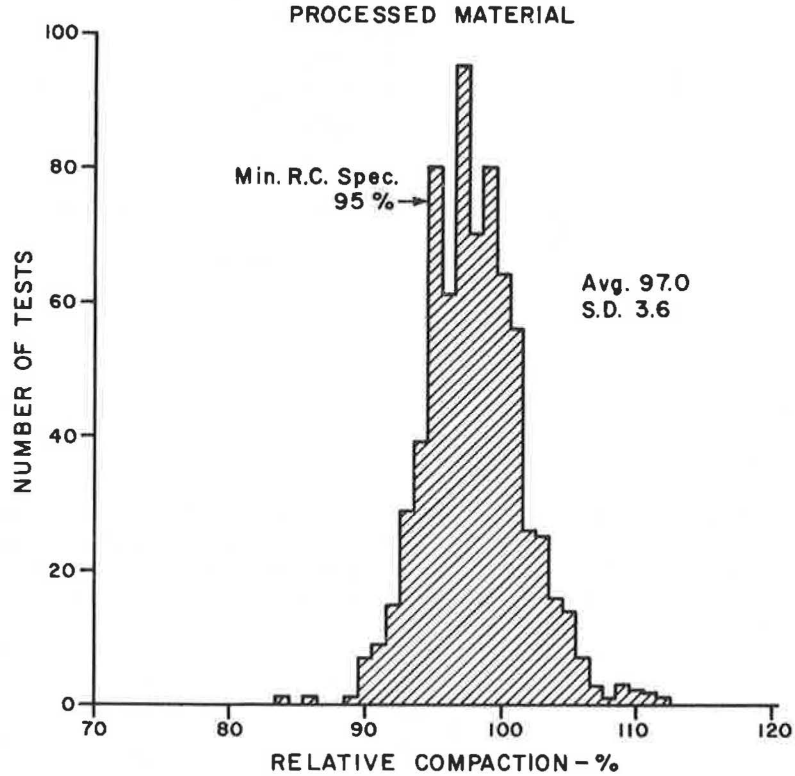

Figure 3. Frequency distribution, Project No. 1—individual test sites, passing areas only.

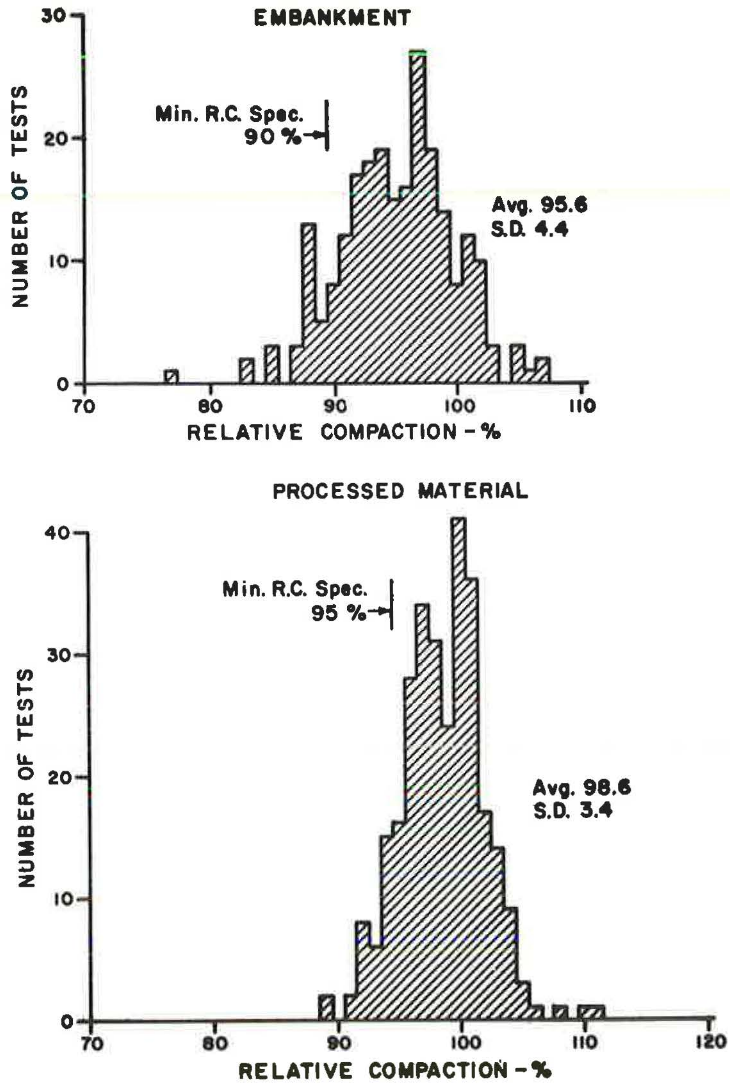

Figure 4. Frequency distribution, Project No. 2—individual test sites, passing areas only.

resident engineers were asked to give it a reasonable trial. They were unanimous in accepting the concept after gaining experience in its use. The acceptance by highway and contractor personnel of this new test method was outstanding, and far exceeded expectations.

It was intended to require compaction equivalent to that previously obtained. The opinions of the various people concerned were that basically no major change in compactive effort has resulted where the new test method was used. However, this is only an opinion and the best comparison would be a study of how the density varied in the accepted earthwork.

Density variations on three completed typical projects are shown in Figures 3, 4 and 5. Project 1 consisted of minor fills and cuts with major structural section work; therefore, only the structural section densities are shown in Figure 3. The range of relative compaction was from 84 to 112 percent with an average of 97 percent and a standard deviation of 3.6 percent. Fourteen percent of the tests were below 95 percent relative compaction.

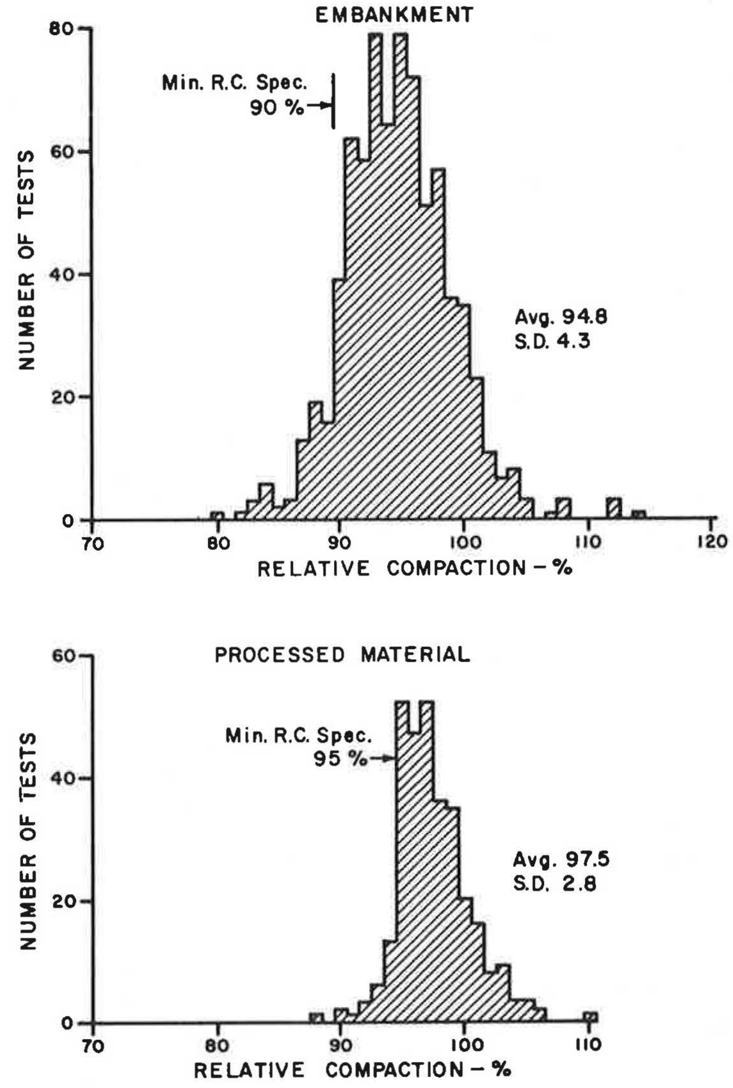

Figure 5. Frequency distribution, Project No. 3—individual test sites, passing areas only.

Project 2 consisted of small cuts and fills in shales and structural section work; test results are shown in Figure 4. About half the embankment material was of such a rocky nature that the sand volume test could not be performed; however, no major difficulty was encountered performing the nuclear tests. The range of relative compaction for the embankment soils was from 77 to 107 percent for an average of 95.6 percent and a standard deviation of 4.4 percent. Twelve percent of the tests were below 90 percent relative compaction. The material with a 95 percent relative compaction requirement is also shown in Figure 4. This material had a range of relative compaction from 89 to 111 percent, for an average of 98.6 percent and a standard deviation of 3.4 percent. Eleven percent of the tests were below 95 percent relative compaction.

Project 3 contained heavy embankment work on soil and rock material; the density distributions are shown in Figure 5. About 30 percent of the embankment material would normally be considered too rocky to test by means of the sand volume equipment; however, all soils were testable with the nuclear equipment. The embankment materials indicated a range of 80 to 114 percent relative compaction with an average of

94.8 percent and a standard deviation of 4.3 percent. Seven and one-half percent of the tests were below 90 percent relative compaction. The material with a specified 95 percent relative compaction had a range of 88 to 110 percent, an average of 97.5 percent with a standard deviation of 2.8 percent. Eight percent of the tests were below 95 percent relative compaction.

Distribution plots of the type shown were maintained on all 11 projects and were similar in construction. The plots indicate that the distribution of relative compactions of accepted areas has a higher average value with a smaller percentage of the values below the specification limit than was found in the statistical studies in California (4). This indicates that the new test method results in a slight increase in the quality of the compaction of earthwork being obtained.

District Personnel

The reaction of the district personnel, both in the field and from an administrative viewpoint, was almost complete acceptance. The general feeling was that "Now we know what compaction we are obtaining." Resident engineers agreed that consideration should not be given to replacing the sand volume test by the nuclear test while still using one test only for acceptance or rejection. The area concept was what they wanted. Realizing that the transition to the new test method would require time, two asked that the sand volume test be substituted for the nuclear test in the new test method so that more projects could take advantage of the area concept.

The contractors were required to produce about the same work on compacting earthwork as previously. However, the contractor was able to make more efficient use of his equipment. The general feeling was that the resident engineers were sure of the quality of the work obtained. The number of areas that had to be reworked appears to be about the same as when using the previous test method.

Field personnel felt that the cost and manpower requirements of the two test methods were about equal, with any time savings in favor of the new test method. This would mean that there would be no large financial savings to the Division of Highways from the standpoint of testing costs.

All districts expressed concern about maintenance problems. In the districts where high downtimes had occurred, there was even the suggestion that two nuclear gages be assigned to each project.

Districts were not reluctant to undertake the administrative aspects of the nuclear gages. This includes the training, maintenance and health safety programs.

The districts expressed general agreement that the new test method should be used on the high-production projects. Some districts felt that "fly" parties could handle the smaller projects. All districts felt that from an administrative viewpoint there should be a gradual transition from the present conventional testing to the new area concept.

Contractors

At the start of the research program most contractors appeared to be neutral; however, by the end of the program the majority were definitely favorable. The favorable reactions appeared to be based on the following points.

Using nuclear gages helped supply quick results to the contractor. On several projects the contractor's foremen accompanied the State personnel making the tests. When a portion of the area would start failing, the contractor's method of operation would be changed. The contractor on one major project was able to control in less than one day his method of compaction of base material. Several contractors cite this rapid obtaining of results as being an important factor in their favoring the new test method.

The new method enabled contractors to utilize their equipment more efficiently. Often a portion of the fill would be below specified compaction and only this portion would need additional compaction. This portion could receive additional compaction while embankment material was placed on the remainder of the fill. Also the contractors often were able to vary compaction patterns so as to obtain reasonably uniform compaction over the entire fill. The contractors readily accepted the results of the area concept and did not question the rejections as had frequently occurred with the sand volume test method. Several contractors expressed approval of the area concept and felt that it greatly aided in the planning of their operations.

An unexpected item was the contractors' reaction to the use of the nuclear gages in relation to their operations. They no longer needed to stop all hauling operations while the sand was being poured during the sand volume test. One contractor estimated that this item alone cost him $25,000 to $50,000 per year.

The contractors estimated that a zero to two cents per cubic yard reduction in cost, depending on conditions, could be expected when the new test method was adopted. However, the actual savings in construction costs is difficult to evaluate because so many other factors influence the bid prices. This saving may be realized at times; however, it is felt that the estimated dollar savings is somewhat indeterminate at present.

Plan of Operation

The implementing of the decision to convert to the new compaction control method will be a gradual process. The modified statistical test method still will be used on the larger earthwork projects that are let to contract each year. At first it is anticipated that each highway district will submit to headquarters the projects on which they wish to use the new test method, and then the available gages could be assigned to the districts where they will be best utilized. Some districts also desire to establish "fly" parties for the smaller projects, which can also be done gradually. Depending on financing, it is estimated that 3 to 5 years would be required to fully equip the various projects with nuclear gages.

Each of California's 11 highway districts will be licensed to handle and administer the use of the nuclear gages, including the health safety, training, and maintenance aspects. The standardization of the test procedure and purchase of the nuclear gages will be handled as a function of headquarters, as on all other testing. Health safety will be handled by the district Radiation Safety Officer, and all health safety and administrative records will be kept at the district level. District personnel will conduct training courses at the district level. Maintenance of the nuclear gages will be performed by service agreement. The Administrative Officer in each district will be responsible for maintenance, and assigning of gages to the individual projects.

The nuclear gages will be purchased by headquarters for the entire state by bid. The nuclear gages will be of the transmission-backscatter type measuring both moisture and density, and will be constructed to California Division of Highways specifications.

The test procedure will be standardized statewide and revisions made periodically as necessary. The test method will be a part of the California Division of Highways Materials Manual with the designation Test Method No. Calif. 231.

CONCLUSIONS

With the completion of 7 years of studies and research on compaction control utilizing nuclear gages, a practical test method has been developed. This test method is a modified statistical method that utilizes nuclear gages to determine the in-place soil moisture and density. A specification for a nuclear gage that will perform satisfactorily has been prepared. It is felt that this new test method utilizing the area concept and nuclear gages represents a definite improvement in compaction control.

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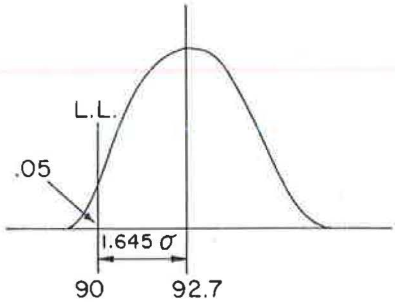
Appendix

Statistical Determination of Sample Size

Assumptions

1. Lower Limit of Specification = 90% R.C.
2. Ave. of all Comp. Results Approx. 92.7% R.C.
   (from earlier studies (-4)
3. Standard deviation of process + 4% R.C.
4. Probability of .95



$$\sigma_{\bar{x}} = \frac{2.7}{1.645} = 1.64$$

$$\sigma_{\bar{x}} \quad \frac{\sigma}{\sqrt{n}}$$

$$\sqrt{n} = \frac{\sigma}{\sigma_{\bar{x}}} = \frac{4}{1.64} = 2.44$$

$$n = (2.44)^2 \doteq \underline{5.96}$$

Use sample size of 6

A Statistical Analysis of Embankment Compaction

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This study statistically examined the distribution of percent relative compaction obtained with current compaction control procedures. The survey included three embankment projects, the soils of which varied from homogeneous to very heterogeneous material. Testing operations for each sampling location included two in-place density determinations by the sand volume method, and two maximim density determinations by the California impact method for each sand volume test.

An analysis of percent relative compaction results for the three projects revealed average values of 92.9, 90.5, and 93.6 percent with standard deviations of 2.4, 3.1 and 5.5 percent, respectively. The greatest dispersion in results was found to exist for the heterogeneous soils.

Factors contributing to the dispersion of percent compaction were found to be the variation inherent in the testing procedure, the soil, and in the compaction process. As the soil becomes more heterogeneous, the effects of variation within the soil and compaction process become more pronounced. This is reflected in the distribution curves for the three projects. Curves are presented which provide a comparison of field control test results and randomly sampled test results. A partial review of problems expected to be encountered in the development and use of purely statistical specifications is presented.

•THE existence of variations in embankment compaction and in the associated control tests has been recognized for a number of years, although many engineers have not been greatly concerned with the extent of variability. The lack of concern regarding variations in test results may be attributed to the type of specification most often employed—which contains a lower limit only. For this type of specification, the dispersion of results is relatively insignificant in relation to construction control procedures.

Highway engineers recently have become more aware of, and interested in, the variability in compaction, due mainly to the efforts of the U.S. Bureau of Public Roads to improve present specifications. The embankment compaction specification used at the AASHO Road Test included the statistical concept of quality control, which also helped to stimulate this interest. However, the use of statistical methods at the Road Test was primarily to insure uniformity of quality in order to better correlate road performance with quality of construction. Therefore, the main objective was to control compaction variation as much as practical rather than to determine variations obtained with usual construction procedures (1).

Data regarding the reproducibility of test methods for measuring in-place and maximum densities have been reported since about 1950. However, except for the works of Davis in 1953 (2) and Carey in 1957 (3), very little information has been published regarding variations in density of compacted embankments. One of the primary purposes of this study is to add to the knowledge concerning the statistical parameters of relative compaction.

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

SAMPLING AND TESTING PLAN

The BPR Outline

The U.S. Bureau of Public Roads, through their regional workshops, presented to state highway department representatives a general outline for statistical surveys. The Bureau then left to the individual states the formulation and execution of sampling and testing plans for those particular items selected by the states for study. The general outline included the following requirements:

1. For each item being considered (in this case embankment compaction control), at least three separate construction projects should be surveyed. These three projects should represent, as nearly as possible, the range of problems and materials encountered throughout the state.
2. At least 50 sampling locations should be randomly chosen for each project.
3. Two samples should be taken at each sample location.
4. Duplicate tests should be made on each of the two independent samples taken from each location.
5. The samples should be taken, as nearly as possible, under normal field conditions by district construction inspectors.
6. The study should be independent of, and in addition to, the normal job testing and control procedures.
7. Only those materials accepted by the resident engineer should be considered in the survey.
8. Whenever possible, ASTM or AASHO test methods should be employed. When necessary to use a test method primarily of local acceptance, a similar ASTM or AASHO test should also be performed.
9. Analyses of test data should include an analysis of variance. This would include a measure of the variance between tests on duplicate samples, the variance inherent in the sampling method, and variance inherent in the material or process.

The duplicate samples from each location provided a measure of the variance inherent in the sampling process. Duplicate tests on each sample provided a measure of the variance inherent in the testing process, and the 50 test locations on each project provided a measure of the basic variance in the process or material.

Modifications of the BPR Outline

The BPR outline was general in nature and could be applied to many construction materials or processes. Because of its generality, certain modifications were necessary for physical reasons. For example, in the case of embankment compaction, the sampling and testing operations are not independent because it is not possible to split a sand volume sample to obtain two independent test results. As a compromise, two in-place density tests were made by the sand volume method at each location (Fig. 1). These tests were taken reasonably close together and never more than 3 ft apart. A sample of the soil was taken for each sand volume test and then was carefully split for maximum density determinations. Thus, at each location results of two sand volume tests and four laboratory maximum density tests were obtained.

On both Projects 1 and 2, 50 locations were sampled; but on Project 3, rain cut short the construction period and only 44 locations were sampled. It was also necessary to depart somewhat from the original request that these samples be chosen in a completely random manner. A true random sampling plan for the entire project would have required the locations to be randomly selected from the entire volume of fill material to be placed on the project, thus allowing each incremental volume an equal chance of being sampled. In accordance with construction needs, and to keep a reasonably uniform flow of work to the testing engineer, it was necessary to randomly choose one location each day from a fill area accepted as satisfactory by the resident engineer. Thus, at the end of the working day the sampler and the grade inspector established those areas or sections which had been tested and accepted by the resident engineer's personnel. The research sampler then determined the total length of these sections

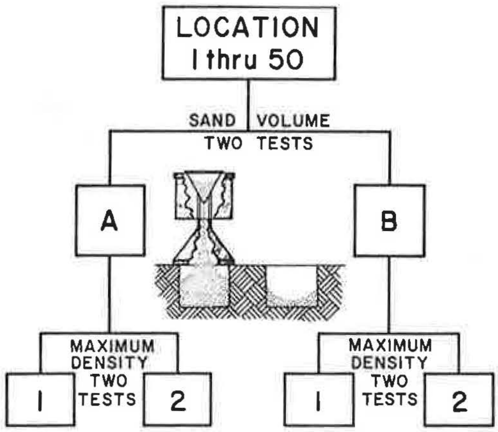


Figure 1. Testing procedure for each sampling location.

and multiplied this length by a random number taken from a table. This established a length which was readily converted to a station location. The sampler then stepped across the fill at this station to determine the width and multiplied this width by the next number from his table, thus establishing a random location for testing. The following day he repeated the process on a new area using the next set of random numbers from his table. This deviates from a true random sample because the areas from which the daily samples were chosen were not of equal size. This seemed, however, an acceptable compromise with the engineering needs. The system worked very well since one man was able to determine the location and do all necessary field and laboratory testing in one day. An example of this random sampling from an area is included in the Appendix.

Test Methods

The percent relative compaction was determined by California Test Method No. 216-F. Since this method is primarily of local acceptance, additional tests by AASHO Test Method No. T180-57C were made to provide both a comparison of results and a check of survey data. Previously reported work showed that the results of the two methods correlate with most types of soils (4).

The primary difference between these two methods is in the laboratory apparatus and procedure. The compaction mold for the California Test Method is 3 in. in diameter and the specimen height varies from 10 to 12 in. Consequently, the volume is variable. The mold of the AASHO method has a diameter of 4 in. and a constant height of $4\frac{5}{8}$ in. The tampers both weigh 10 lb and free drop 18 in. Both methods utilize 5 layers. Each layer is subjected to 20 blows in the California method and 25 blows in the AASHO method. The resulting compactive energies are approximately 33,000 and 56,250 ft-lb/cu ft for the California and the AASHO methods, respectively. Maximum densities by both methods are determined from that portion of material passing the $\frac{3}{4}$-in. sieve. Corrections for larger size material are applied to results from the California method if the percentage of larger sizes exceeds 10 percent. No corrections are applied to results from the AASHO method.

The California Test Method does not require the determination of moisture content unless the correction in unit weight is made for oversized material. As a result, information regarding moisture content was not always available. Results presented here are therefore based on wet unit weights.

PROJECT DESCRIPTIONS

The three contracts in this survey were major projects on divided four-lane highways. Projects 1 and 3 closely approximated the smallest and largest variation in soil characteristics normally expected in California. Project 2 was somewhere between these two extremes. Typical grain size distribution curves are shown in Figure 2. Table 1 includes sieve analyses, liquid limits, plastic limits, plasticity indices, and sand equivalent values.

Project One

The embankment material on Project 1 was primarily a highly decomposed granite, weathered to a clayey silty sand of medium plasticity. Embankments consisted of

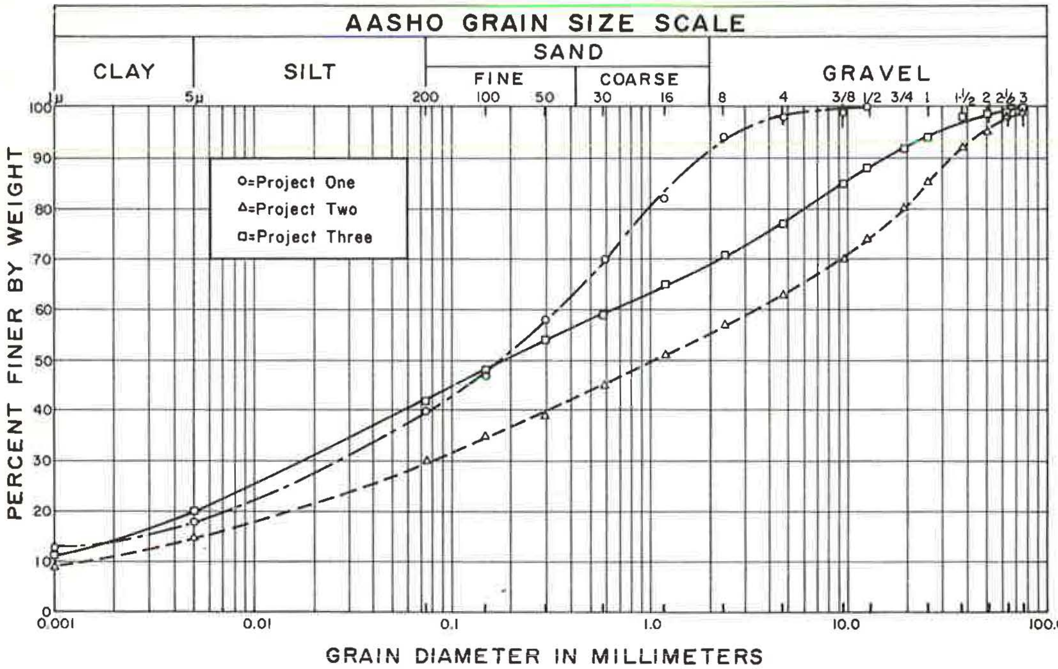


Figure 2. Grain size distribution curves.

TABLE 1

INDEX PROPERTIES AND GRADATION OF EMBANKMENT SOILS

Identifying Properties	Project 1		Project 2		Project 3	
	Avg.	Range	Avg.	Range	Avg.	Range
Sieve analysis (% passing)						
3 in.			99	97-100	100	
2½ in.			98	96-100	99	99-100
2 in.			95	92-100	99	98-100
1½ in.			92	85-96	98	96-99
1 in.			85	78-91	94	91-97
¾ in.	100	99-100	80	73-88	92	86-95
½ in.	100	98-100	74	66-84	88	79-93
⅜ in.	99	97-100	70	62-81	85	72-91
No. 4	98	96-99	63	55-77	77	59-88
No. 8	94	89-97	57	48-69	71	52-80
No. 16	82	76-88	51	43-63	65	45-76
No. 30	70	62-77	45	37-59	59	38-70
No. 50	58	50-66	39	32-53	54	33-65
No. 100	47	40-54	35	28-47	48	27-59
No. 200	40	34-49	30	24-43	42	23-54
5 micron	18	15-20	15	10-23	20	11-27
1 micron	13	12-14	9	6-15	11	3-17
Sand equivalent	15	12-17	12	9-15	10	7-17
Liquid limit	25	21-28	33	32-34	29	22-34
Plastic limit	15	14-16	21	19-22	17	15-20
Plasticity index	10	7-13	12	11-13	12	7-17

Number of tests made to determine above items: Project 1, 10 tests per item; Project 2, 10 tests per item; Project 3, 7 tests per item.

shallow fills across valley terrain. Most of the main line fills were only 2 or 3 ft in height with one short section reaching 14 ft. The soil was in a fairly dry natural state and water had to be added. The project was 4.1 mi in length and the total embankment involved only 350,000 cu yd.

Project Two

Project 2 was in a region of rolling terrain where the material was predominantly a medium plastic red clayey silty sand containing lenses of stream-rounded aggregate with cobbles up to 6 in. in diameter not uncommon. Some aggregate was well dispersed throughout the fines, and it was possible to excavate this material without blasting.

The total length of the project was 5 mi and the height of the embankments varied from 3 or 4 ft to a maximum of 26 ft. A total of 1,200,000 cu yd of embankment was placed on this project.

Project Three

Project 3 was in the Franciscan Formation, which is characterized by landslides as well as erosion. Many of the landslides in this area are still active and the slip surfaces are characterized by wet, low-strength material. Even some of the harder materials had a relatively high moisture content, a common characteristic of the sandstone and sheared shale of the Franciscan Formation. Blasting was often required during excavation. Some fills were so rocky that they had to be excluded from this study, while others were predominantly clay and silt. On some fills the contractor found it necessary to blend dryer materials with the wet, heavy clays in order to achieve a satisfactory water content. The project was 4.5 mi in length and had 1,760,000 cu yd of embankment. Height of fills varied from 3 to 38 ft.

DISCUSSION OF RESULTS

California Impact Test vs AASHO Test T 180-57C

In addition to the statistical survey tests, which were performed by district personnel according to the California test procedure, further tests for maximum density were made by Headquarters Laboratory according to both the California and AASHO procedures. These additional tests were made to provide a means for comparing maximum density results as obtained by the two methods, since the California method was primarily of local acceptance.

On Project 1, two tests by the AASHO Test Method T180-57C were made at each of 26 sampling locations. These were performed at the job site by Headquarters Laboratory personnel. On Projects 2 and 3, ten and seven sampling locations were selected, respectively, and material was shipped to the Headquarters Laboratory for testing.

TABLE 2

COMPARISONS OF MAXIMUM DENSITY DETERMINATIONS
(California 216-F vs AASHO T180-57C)

Computed Quantity	Project		
	1	2	3
Number of Locations	26	10	7
Avg. Dist. (Calif. method)	141.5	140.2	147.7
Avg. HQ (Calif. method)	—	140.7	146.6
Avg. HQ (AASHO method)	140.8	141.8	147.0
Average Difference:			
Dist. results (Calif. method) minus HQ results (Calif. method)	—	+ 0.5	- 1.0
Dist. results (Calif. method) minus HQ results (AASHO method)	+ 0.72	+ 1.6	- 2.1

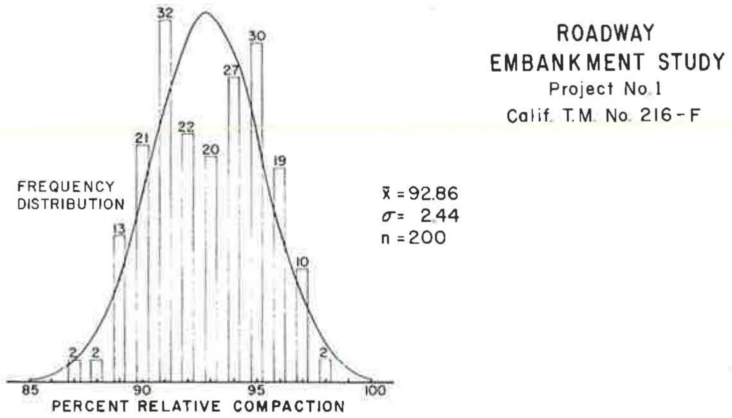


Figure 3. Relative compaction histogram, Project 1.

One maximum density determination by each method was performed on the material shipped from each sampling location. Results of these tests are given in Table 2.

For all practical purposes, the California and AASHO methods produce approximately equal average results. However, this conclusion is based on a small number of tests. The findings are in agreement with earlier published information (4).

Percent Relative Compaction

The percent relative compaction distributions for the three projects are shown in Figures 3, 4, and 5. Each figure consists of a frequency histogram of the actual survey data and a normal curve. The normal curve for a particular project represents the most probable distribution for all possible test results from that project. No explanation other than random variation was found for the bimodal distribution shown in Figure 3 or the non-normal distribution shown in Figure 5.

The plots show significant differences in the dispersion of relative compaction results for the three projects. This dispersion of compaction values about their average could be due to several factors, all of which affect both maximum and in-place density values. These factors include the variation in soil properties and the nonuniformity of field compaction conditions within the area tested. For example, local variations in the soils of Projects 1 and 2 were appreciably less than the variation for Project 3.

A portion of the dispersion may be inherent in the basic testing process. In-place and maximum densities of a particular soil are related; therefore, the practice of expressing one as a percentage of the other would seemingly compensate for variations

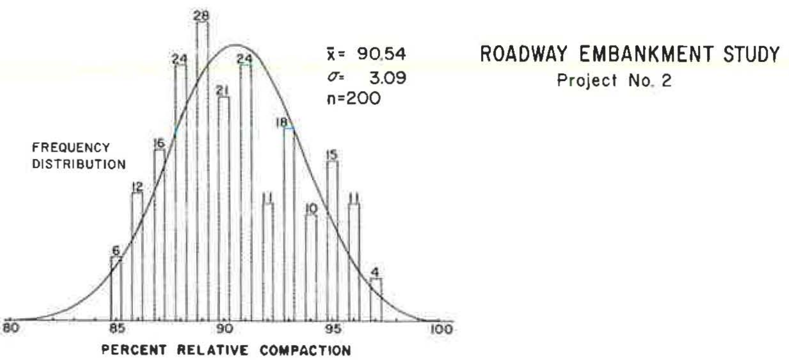


Figure 4. Relative compaction histogram, Project 2.

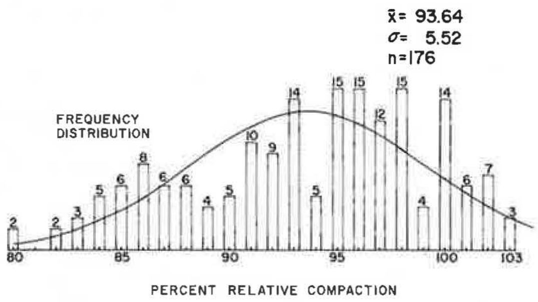


Figure 5. Relative compaction histogram, Project 3.

in magnitudes of the two and result in a fairly constant value for relative compaction throughout a project. In many instances, however, laboratory compaction is not entirely representative of field compaction. Consequently, values for the two densities often do not change in the same ratio, even within small areas for highly variable soils, thereby causing variations in relative compaction values.

The effects of soil variations on compaction were observed in 1953 by Davis (2). His statistical findings were from 29 construction locations on 23 earth dam projects. Davis reported standard deviations ranging from approximately 1.8 to 5.0 percent relative compaction with all 29 locations averaging 3.3 percent. The standard deviation for embankment soil under flexible pavement sections of the AASHO Test Road was approximately 1.85 percent relative compaction (1). This low standard deviation, however, was obtained with much more rigid control and a greater number of tests than would be economical for normal construction projects. Another factor contributing to the low standard deviation was the extremely uniform soil used on the Test Road. The standard deviation reported by both Davis and AASHO are in general agreement with the standard deviations determined for the three projects reported here.

The percentages of tests in this study failing to meet the minimum compaction requirement are comparable to previously reported data. Results from the AASHO Road Test, for example, indicate that approximately 8.5 percent of all embankment material tested failed to meet the lower specification requirement (1). Statistical estimates indicate that the percentages of failures in Davis' data vary from about 10 to 25 percent, with a few as high as 45 percent.

Numerical values from Figures 3 through 5 are summarized in Table 3. All values were computed from the special survey data only. The data in Table 3 illustrate the dependence of percent failing on the relationship between average and standard deviation.

TABLE 3

SUMMARY OF RELATIVE COMPACTION RESULTS

Quantity or Characteristics	Project		
	1	2	3
Number of sampling locations	50	50	44
Number of relative compaction determinations	200	200	176
Range of relative compaction results (percent relative comp.)	87-98	85-97	80-103
Average compaction (percent relative comp.)	92.9	90.5	93.6
Standard deviation (percent relative comp.)	2.4	3.1	5.5
Percentage of compaction tests less than spec. limit of 90 percent relative compaction	8.5	43.0	23.9

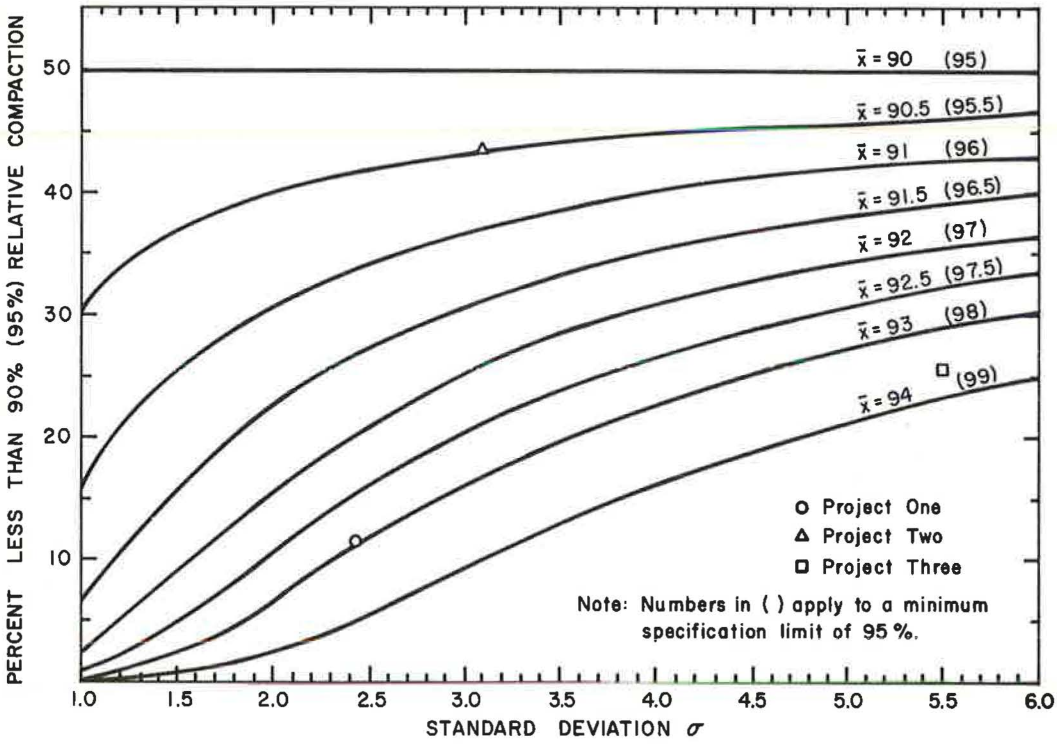
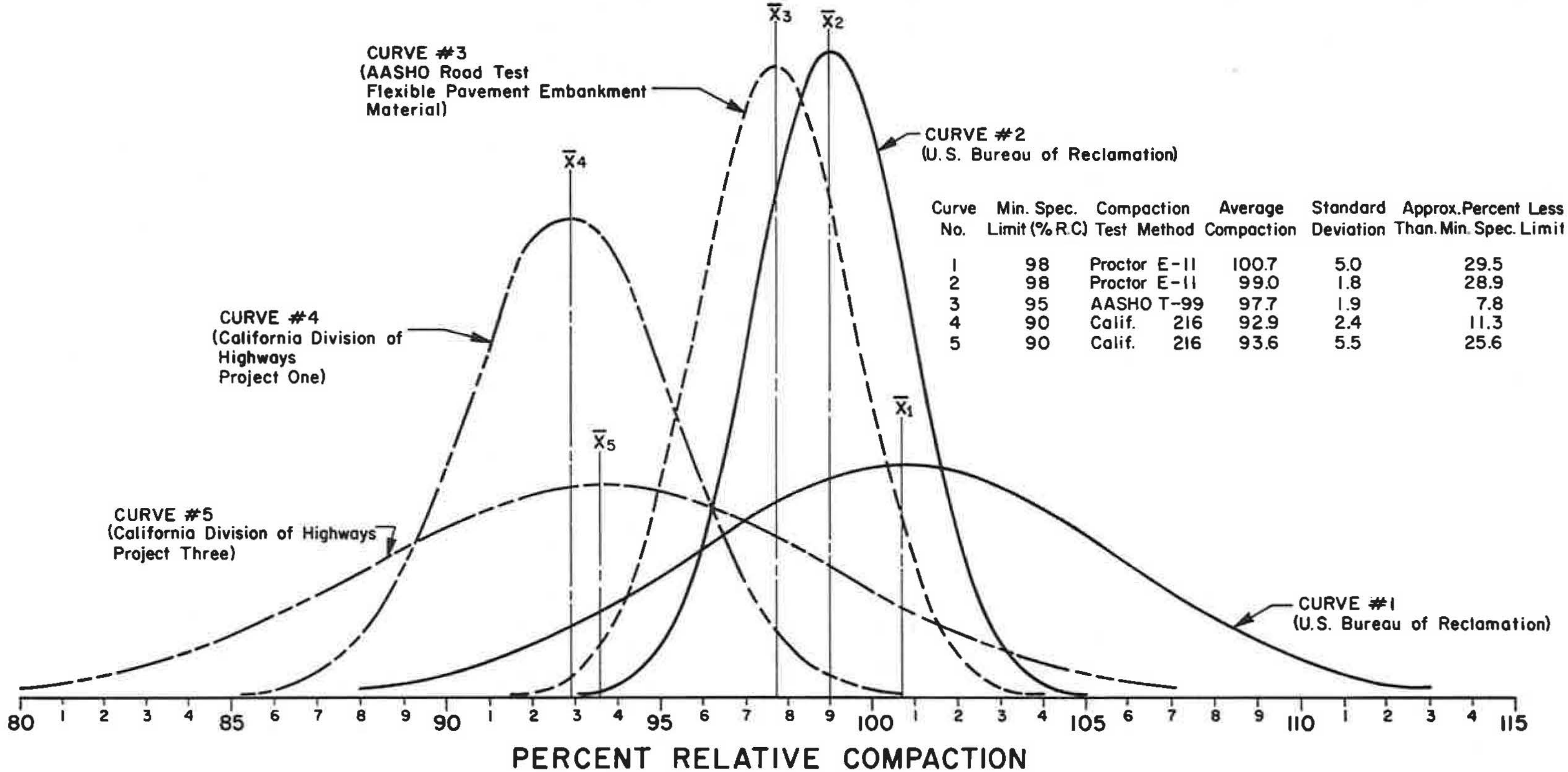


Figure 6. Relationship between standard deviation, average and percent less than lower compaction limit.

For example, comparing the values of the average, standard deviation, and percent failing for the projects shows that the percentage of failures tends to decrease with an increase of average, and increase with an increase of dispersion, as measured by the standard deviation (Fig. 6). The curves in Figure 6 show the percent failing plotted against standard deviation, with average as a parameter. Although the curves are theoretical in nature and are based on normal distributions, they produce values very close to those in Table 3 for the three projects surveyed.

Figure 6 shows that with the three soils tested with the measured standard deviations, the following overall averages would have to be obtained, if no more than 10 percent of the finished embankment is to be below the 90 percent specification limit: Project 1, 93 percent; Project 2, 94 percent; and Project 3, 97 percent. Within the limits of experimental error, Project 1 meets this criteria; however, the average values for both Projects 2 and 3 would have to be increased by 3 percent, if no more than 10 percent is to be below specifications.

Figure 7 shows some selected normal distribution curves superimposed on the same scale. This illustrates to some extent the relative dispersion in percent compaction for projects of different organizations. Curves 1 and 2 were prepared from the data reported by Davis (2) and represent two of the 29 construction locations. The minimum specification limit for this earthwork was 98 percent relative compaction, as determined by the Bureau of Reclamation's Standard Proctor Compaction Test Designation E-11. Curve 3 represents the AASHO Road Test embankment material for flexible pavement sections. Upper and lower specification limits of 100 and 95 percents were employed for this project. Maximum densities were determined by AASHO Test Method T-99. Curves 4 and 5 are those of Projects 1 and 3 of this study (Fig. 3 and 5). The minimum specification limit for these latter two curves was, of course, 90 percent using the



Curve No.	Min. Spec. Limit (% R.C)	Compaction Test Method	Average Compaction	Standard Deviation	Approx. Percent Less Than. Min. Spec. Limit
1	98	Proctor E-11	100.7	5.0	29.5
2	98	Proctor E-11	99.0	1.8	28.9
3	95	AASHO T-99	97.7	1.9	7.8
4	90	Calif. 216	92.9	2.4	11.3
5	90	Calif. 216	93.6	5.5	25.6

Figure 7. Comparison of normal distribution curves from three organizations.

TABLE 4

EFFECTS OF AVERAGING INDIVIDUAL READINGS

Project	Results of Individual Determinations			Results After Averaging		
	n	$\overline{X}$	σ	n	$\overline{X}$	σ
1	200	92.9	2.4	50	92.9	2.2
2	200	90.5	3.1	50	91.0	3.0
3	176	93.6	5.5	44	93.7	4.5

California method. It is interesting to note in comparing curves 1 and 2 with curves 4 and 5 that the flatter curves, representing greater dispersion, have higher averages than the corresponding steeper curves.

When comparing the curves in Figure 7, it should be noted that they were calculated from the data of three organizations, all of which have different specifications and test procedures. This factor, plus the fact that the soil characteristics for some of the projects represented by the curves are not known, greatly limits the ability to make meaningful direct comparisons between the curves. However, Figure 7 does indicate that various agencies are obtaining similar variations in embankment compaction results for widely different material types.

The effects of averaging the in-place and the maximum density values on the resulting relative compaction distribution are given in Table 4. These relative compaction results were computed from the average of two in-place densities and the average of four maximum densities per location.

The reduction in dispersion given in Table 4 would be an important consideration in the enforcement of specifications. For example, for Project 1 a range of three standard deviations, ±6.6, would be acceptable providing tests were averaged as detailed to obtain the acceptance value. On the other hand, if acceptance is to be based on a single test, a range of ±7.2 must be established to allow for the wider dispersion.

Precision of Test Method

Distributions of maximum and in-place density test data are shown in Figures 8 through 20. Figures 8 through 13 and 17 through 20 show frequency histograms of all the survey density test results. Figures 14 through 16 show maximum density and percent relative compaction against roadway stationing.

The histogram of maximum densities for Project 1 (Fig. 8) exhibits a concentration of test results within the 140-148 lb/cu ft range, which results in a skewed distribution that appears to have been constructed from two distinct sets of data. The explanation for the skewness was found in Figure 14, which shows two distinct soil types located at different stations along the roadway. Test results for both maximum and in-place densities were separated into two groups each, based on Figure 14, and plotted as histograms in Figures 17 through 20.

Figures 17 and 18 are easily recognized as the two parts of Figure 8. No similar breakdowns of soil type by location for Projects 2 and 3 were observed from Figures 15 and 16. These figures show the test results to be dispersed appreciably, but the range and approximate mean appear to be fairly constant throughout both projects.

The maximum density plots for Projects 2 and 3 (Figs. 9 and 10) reveal a wide range in test values for both projects. The histogram for Project 2 approaches a normal distribution, although the same cannot be said for Project 3.

The local scatter in maximum density values may be taken as a good indication of the variation in soil homogeneity when comparing different projects or areas. For example, the 8 lb/cu ft range between stations 390 and 431 for Project 1 (Fig. 17) represents a relatively homogeneous soil when compared to the 28 lb/cu ft range for Project 3. Figures 11 through 13 show the distributions of in-place densities to have even greater dispersion than the maximum densities.

It should be pointed out again that, since the California Test Method usually does not require that the moisture content be determined, all densities are recorded as wet weight. It is realized that dry weight determination would provide additional information, but it is observed that wet weights do provide a good comparison of the uniformity of the three projects.

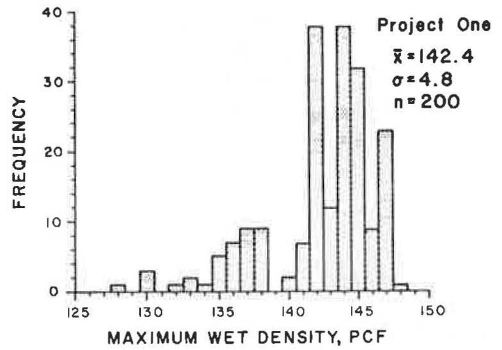


Figure 8. Maximum density histogram, Project 1.

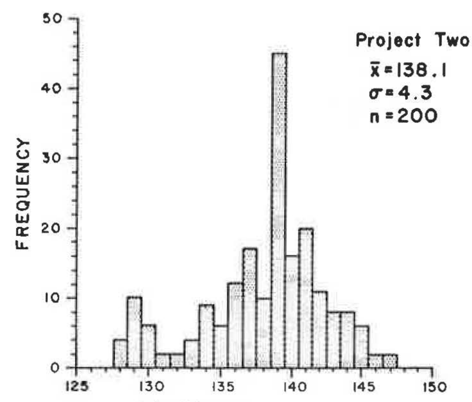


Figure 9. Maximum density histogram, Project 2.

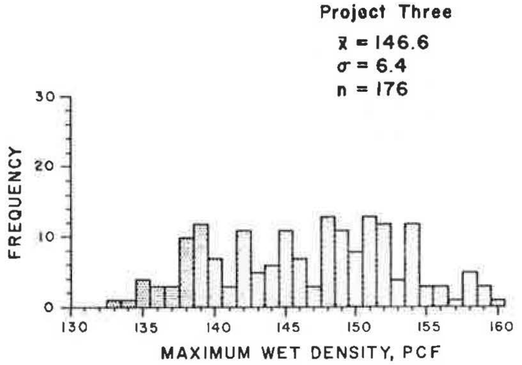


Figure 10. Maximum density histogram, Project 3.

In-Place Density Variations

The maximum recorded variations between two in-place density tests from the same location were 6, 10, and 28 lb/cu ft for Projects 1, 2 and 3, respectively. Actually the one variation of 28 for Project 3 was probably due to some assignable cause and a more realistic maximum would be the next lower observation, i.e., 18.

On Project 1, a shallow fill 4,100 ft long (station 390 to 431) was constructed with unusually homogeneous soil. Thirty-eight pairs of duplicate sand volume tests were performed on this one fill. The standard deviation of the variation between these adjacent sand volume determinations was 2.25 lb/cu ft. This means that 95 percent of the time in this type of soil we could expect the results of two sand volume tests in juxtaposition to agree within the limits of 4.5 lb/cu ft. These results are in close agreement with one study performed on carefully processed uniform soils (8).

On Project 3, where the material was extremely heterogeneous, the standard deviation of the variations between adjacent sand volume tests was 5.96 lb/cu ft. Therefore, for this type of soil, we can expect the results of any two adjacent sand volume tests to agree within the limits of 11.9 lb/cu ft 95 percent of the time.

On Project 2, where the variability of the material is somewhere between the other two projects, the standard deviation of the variation between adjacent sand volume tests was 3.13 lb/cu ft. For this type of soil we can expect results of two adjacent sand volume tests to agree within the limits of 6.3 lb/cu ft.

Remember that these seemingly large variations include not only the inaccuracies within the sand volume test itself, but also the variation in the material and compaction process within the small areas from which the pairs of adjacent tests were taken.

Mention should be made that the California Division of Highways has conducted experimental investigations into the use of nuclear testing equipment. It has been generally concluded that the nuclear equipment has about the same reproducibility as the sand volume test. One study indicated that nuclear surface gage readings could be reproduced within 9 to 10 lb/cu ft

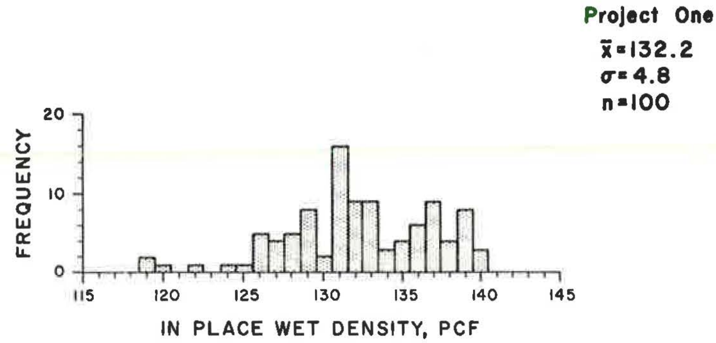


Figure 11. In-place density histogram, Project 1.

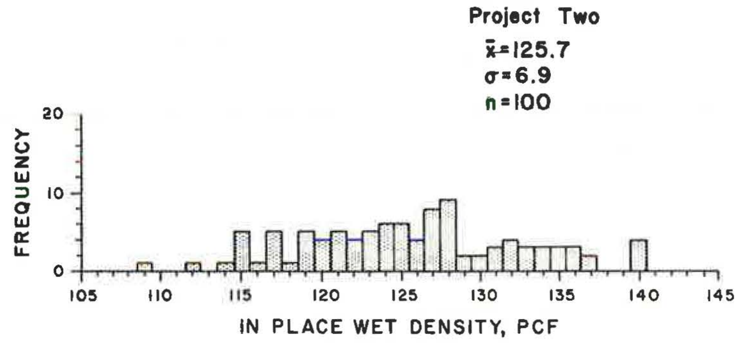


Figure 12. In-place density histogram, Project 2.

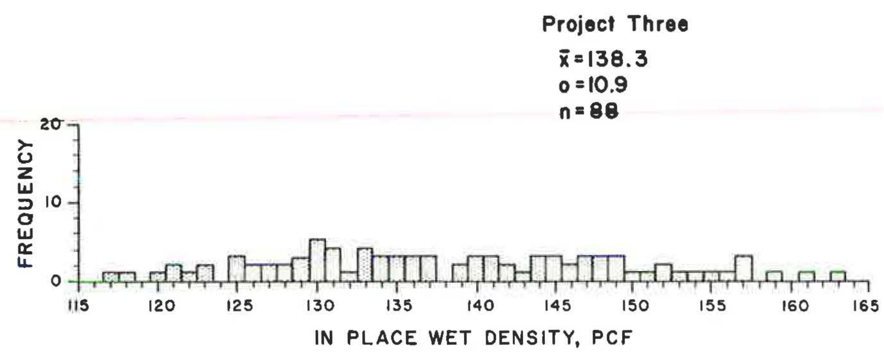


Figure 13. In-place density histogram, Project 3.

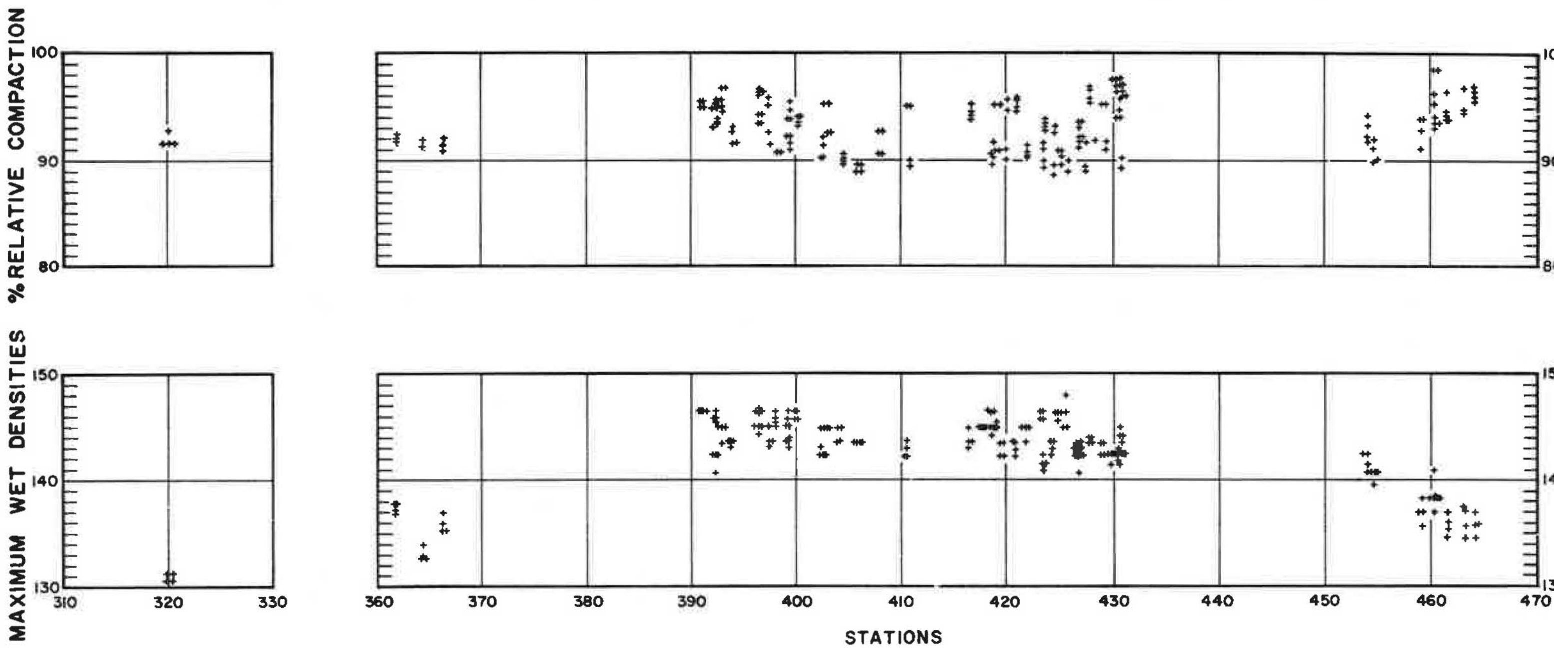


Figure 14. Dispersion of maximum density and relative compaction values related to roadway stationing (Project 1, n = 200).

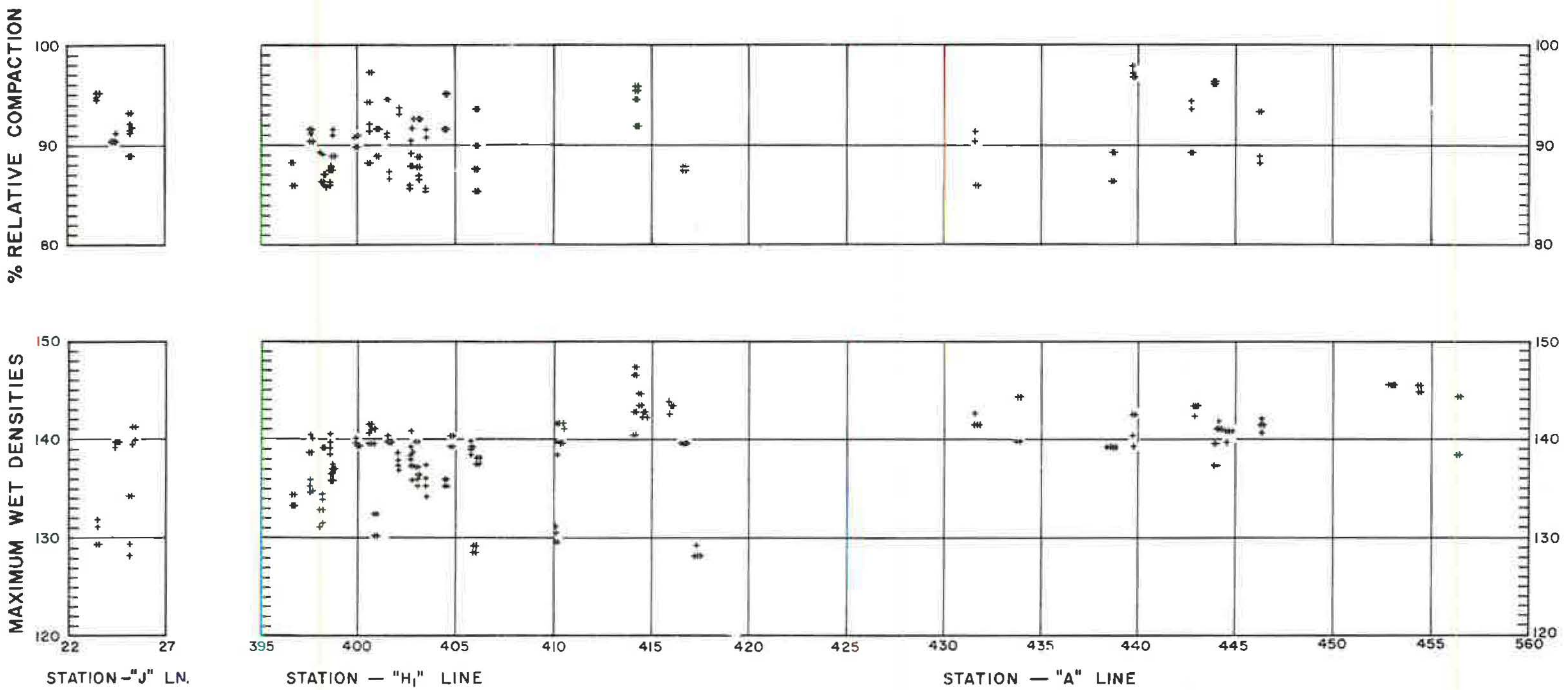


Figure 15. Dispersion of maximum density and relative compaction values related to roadway stationing (Project 2, n = 200).

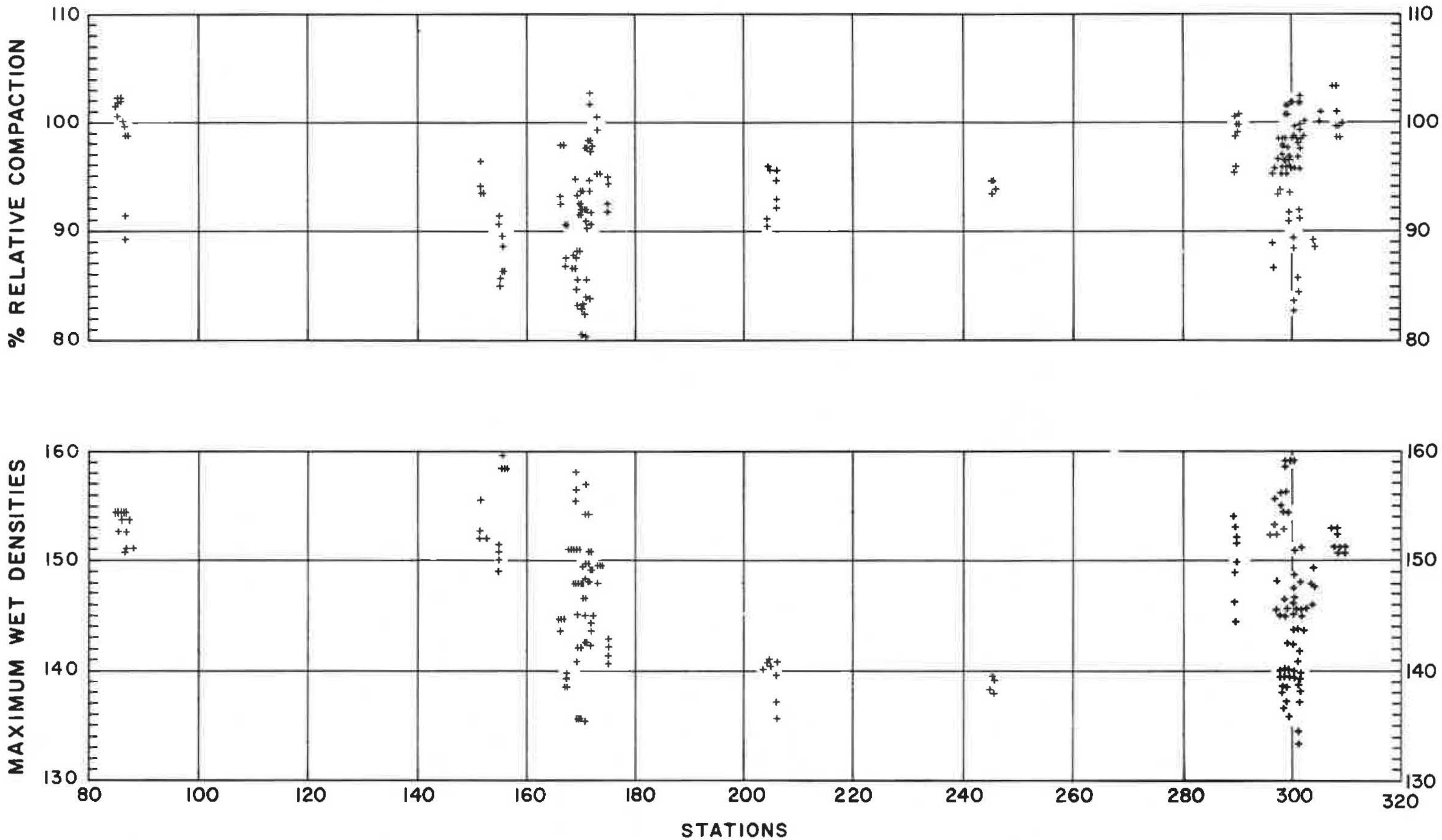


Figure 16. Dispersion of maximum density and relative compaction values related to roadway stationing (Project 3, n = 176).

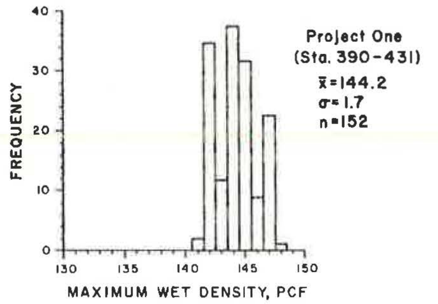


Figure 17. Histogram of maximum density by soil type, Project 1, stations 390-431.

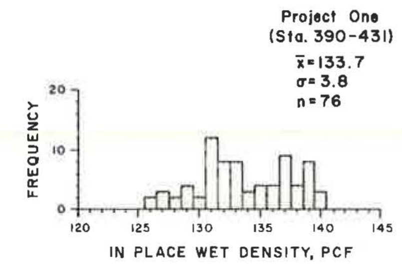


Figure 18. Histogram of in-place density by soil type, Project 1, stations 390-431.

90 percent of the time (11). A later study showed test results from the Lane-Wells Road Logger, a continuously recording mobile nuclear device, to be reproducible within 5 lb/cu ft, while the sand volume test was reproducible within 4 lb/cu ft on the same material. Both of these statements were at a 90 percent confidence level (12).

Maximum Density Variation

An analysis of the survey data revealed that the maximum density test had a standard deviation between carefully split samples of 0.6 and 1.2 lb/cu ft for the materials in Projects 1 and 3, respectively. We can then say that for the materials in Projects 1 and 3 the maximum density test was accurate within 1.2 and 2.4 lb/cu ft, respectively, 95 percent of the time. These values appear to be in very good agreement with previously published data (9).

The standard deviation for Project 2 was only 0.37 lb/cu ft. Further analyses indicated that this low value was due to an assignable cause and was not a true measure of the difference between split samples. It is concluded, however, that the individual determinations of maximum density are reasonably accurate, thus assuring the validity of the overall distribution of the relative compaction determinations.

Relative Compaction Variation

Although the variations in the in-place and maximum density test results would appear to cause very large variations in percent compaction values, such is not necessarily the case. The variations in density test results cannot be added directly. They must be combined according to the probability of occurrence (14). When the values obtained from the computer were combined in this manner, it was found that for the one fill in Project 1, two adjacent determinations could be expected to agree within 3 percent relative compaction 95 percent of the time. For Project 3, two adjacent

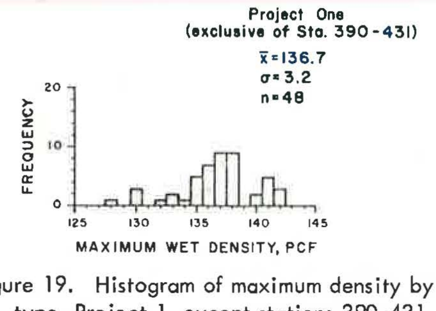


Figure 19. Histogram of maximum density by soil type, Project 1, except stations 390-431.

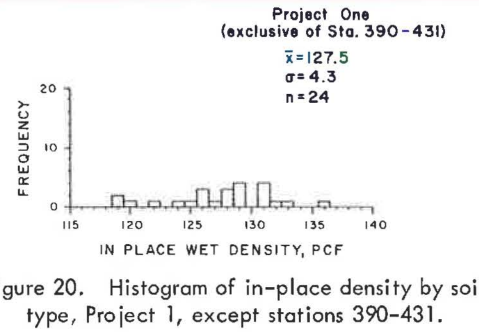


Figure 20. Histogram of in-place density by soil type, Project 1, except stations 390-431.

determinations could be expected to agree within 7 percent relative compaction 95 percent of the time. Note that the previous statements apply only to adjacent tests or tests made within a small area, such as a sampling location used in the survey reported here.

CHARACTERISTICS OF PRESENT FIELD CONTROL PROCEDURES

Effects of Resampling

The accepted practice of only resampling and retesting at locations which fail to meet specifications increases the probability of obtaining a test result within specification limits. This effect may be explained by a hypothetical example as shown in Figure 21. For simplification, it will be assumed that the decision to pass or require additional work will be based on one test result, and further that the distribution of percent compaction of the particular lift being considered is such that 40 percent of all possible test results would be above the minimum specification limit. Thus, the probability of one test result falling below the specification limit is 60 percent.

If the test result falls below the specification limit, some action, such as rerolling, may be taken after which another test is made. Assuming that the additional work has altered the distribution of percent compaction of the lift to the extent that 50 percent of all possible test results would be above the specification limit, a retest would now have a 50 percent chance of passing.

The total probability of accepting this fill based on this sampling and reworking procedure must be obtained from the probabilities of both the first and second tests. Therefore, in this example, the total probability of the second test result passing is 70 percent (Fig. 21).

The example is very similar to the usual procedure in actual practice. If the initial test result is only slightly below the lower limit, a check test is sometimes made. If it is considerably below the lower limit, the contractor would be asked to perform some additional work. Even with additional rolling the soil density may be altered only to a limited extent and the resampling procedure is still affecting the probability of the lift passing.

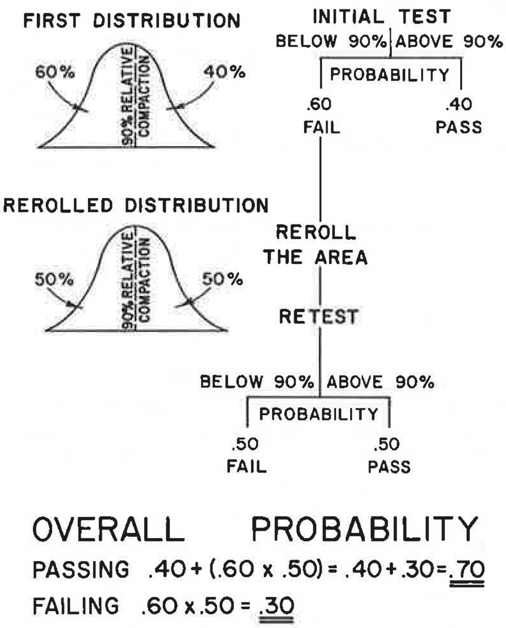


Figure 21. Probability of acceptance considering resampling.

Compaction control differs sharply from the control of those items that can be sampled, evaluated, and then accepted or rejected. The construction of an embankment is often a process of working, sampling, and reworking. Complete rejection occurs only when the material is removed from one or more lifts within a fill. In such instances, the state of compaction is rejected instead of the soil, which may be recompacted under more favorable conditions.

The effect of resampling may be considered in conjunction with the question of how much reworking is required before a new universe is prepared from which a new sample may be drawn. When a lift is rerolled, a new universe is not created; the present universe is merely altered. Although the alteration may be very small, it is nearly always an improvement. In a situation requiring rerolling, the effect of resampling is still present. Here lies one of the problems in present methods of field compaction control. This also explains the discrepancy between field control records and accurate statistical estimates of the state of compaction. There

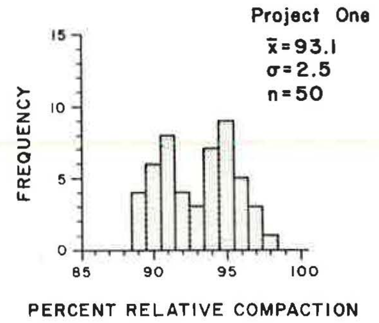


Figure 22. Relative compaction, Project 1, initial test.

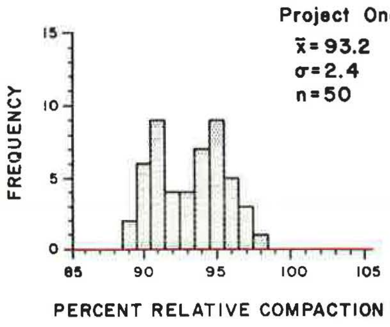


Figure 23. Relative compaction, Project 1, first retest.

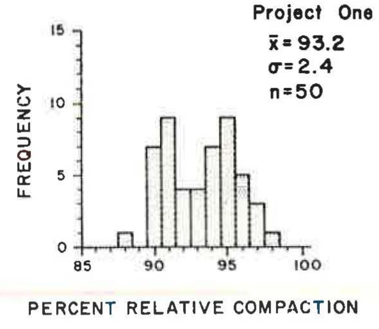


Figure 24. Relative compaction, Project 1, second retest.

appears to be no practical way of limiting the number of times an area may be rerolled and resampled.

The effect of resampling may be illustrated further by considering the data obtained from this study. Only Project 1 will be considered since this amply makes the point. Since two maximum density determinations were made for each of the two in-place density determinations per sampling location (Fig. 1), four individual percent-compaction values were obtained for each location.

To represent a field control procedure, one of the four determinations was randomly selected from each location. These values were plotted as a frequency histogram (Fig. 22). Those locations having percent compaction values less than 90 were then retested. The retesting procedure consisted of eliminating the previously selected failing values, and randomly choosing another value from the remaining three at that location. These new values were combined with the passing values of the initial selection and plotted as frequency histograms (Fig. 23). The procedure was repeated until the fourth value was used for those locations still yielding values of less than 90 percent (Fig. 24).

The initial selection or test resulted in 8 percent of the results being less than 90 percent compaction. This value was reduced to 4 percent by the first retest and to 2 percent by the second retest. The third retest, utilizing the fourth value per location, produced no further reduction in percent failing. In fact the third retest for Project 1 resulted in a histogram identical to that of the second retest. The results of these procedures for Project 3 are shown in Figures 25 through 28.

Project Control Data

Job control records for the three projects were reviewed and compared to the statistical study data to determine the effects of representing the universe by different sets of data obtained under different field conditions. The universe, in this case, is the state of embankment compaction. The different field conditions producing three sets of data include (a) the initial job control tests, which were all the first control tests whether acceptable or not; (b) the final control tests, which include all acceptable initial control tests and the last test of each series of retests made after additional rolling; and (c) the random survey data, which were obtained after the work was accepted by the resident engineer.

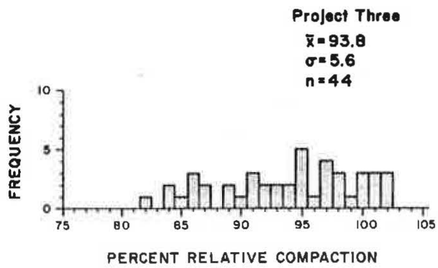


Figure 25. Relative compaction, Project 3, initial test.

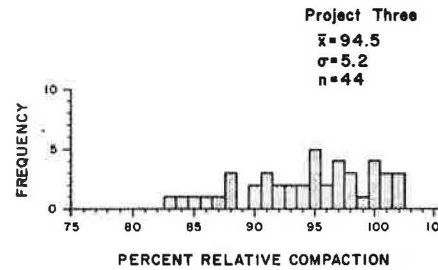


Figure 26. Relative compaction, Project 3, first retest.

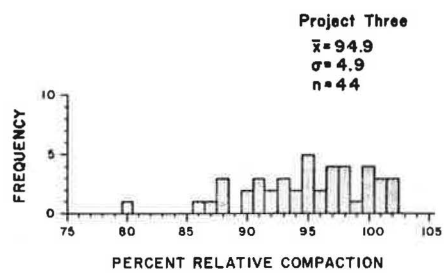


Figure 27. Relative compaction, Project 3, second retest.

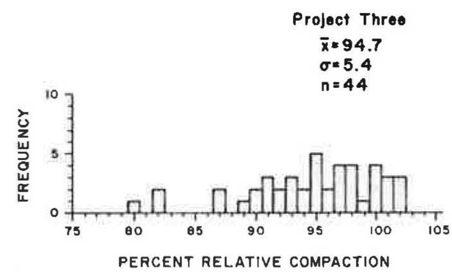


Figure 28. Relative compaction, Project 3, third retest.

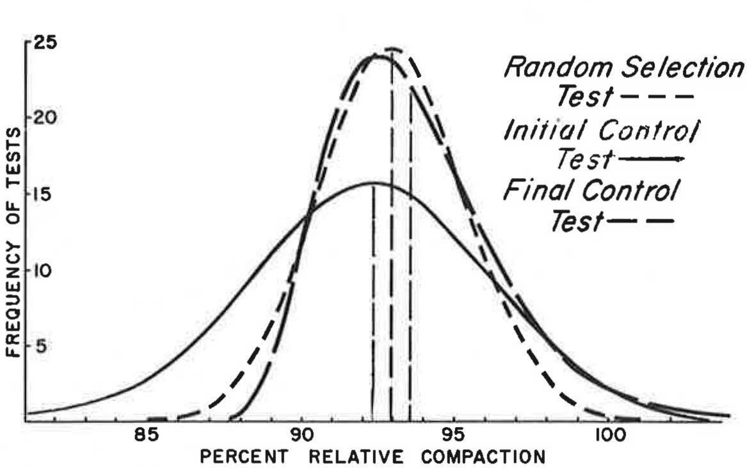


Figure 29. Random sample curves vs project control curves, Project 1.

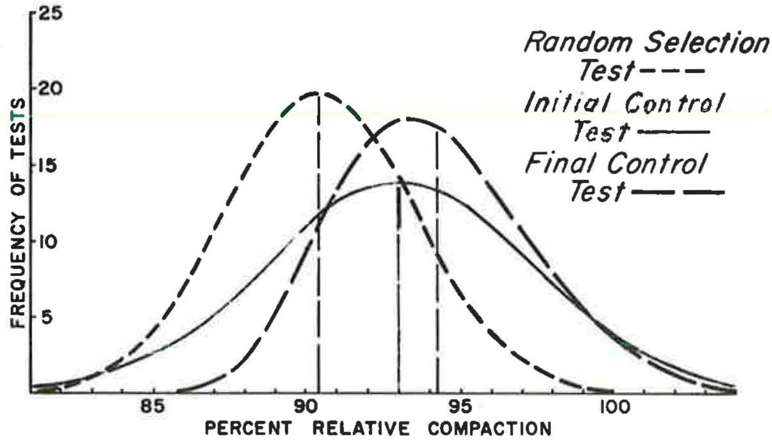


Figure 30. Random sample curves vs project control curves, Project 2.

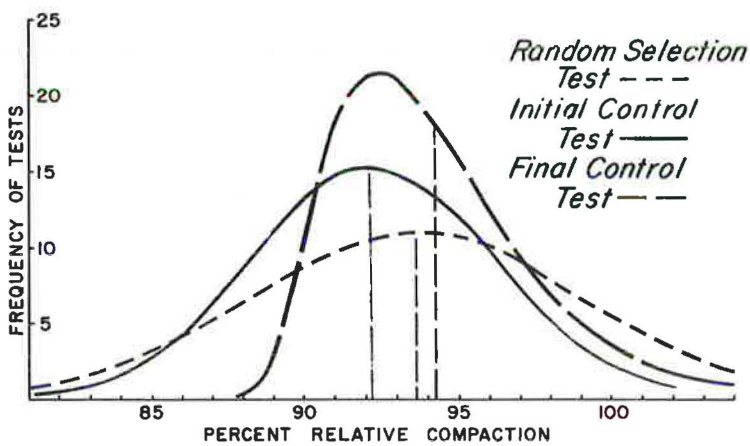


Figure 31. Random sample curves vs project control curves, Project 3.

TABLE 5

PERCENTAGE OF TEST VALUES BELOW 90 PERCENT RELATIVE COMPACTION

Tests	Project		
	1	2	3
Initial project control	23.8	17.6	25.2
Final project control[a]	1.2	2.4	1.0
Randomly located	8.5	43.0	23.9

[a]Final control tests are the last retests or the initial tests if no rerolling was required.

The differences in field conditions were primarily those with regard to rolling. For example, random survey tests were performed subsequent to all rolling operations. However, the final job control tests were not performed at randomly selected locations and the results include the effects of resampling. Initial job control tests were performed after a certain amount of rolling appeared to be sufficient, based on the judgment of the engineer and the contractor. Thus, the random survey data

TABLE 6

COMPARISON OF STATISTICAL STUDY TO INITIAL PROJECT CONTROL TESTS

Item	Project					
	1		2		3	
	Stat. Study	Initial Control	Stat. Study	Initial Control	Stat. Study	Initial Control
No. of tests	200	164	200	125	176	103
Average	92.86	92.30	90.54	93.10	93.64	92.11
Standard deviation	2.44	3.87	3.09	4.37	5.52	3.98

and the final job control data have the advantage of more rolling than the initial job control data. The final control data further have the advantage of including the effects of resampling.

The distribution curves representing the universe under the three different conditions are shown in Figures 29 through 31. These figures show that the largest average for all three projects is obtained from the final job control tests. This would be expected, based on the conditions previously stated. The curves for this set of data for all three projects are skewed to the right; i.e., the left side is relatively steep while the right side tails off. This characteristic is the result of rerolling and resampling which, in this case, eliminates extremely low values. Due to the elimination, or reduction in prominence, of the left tail of the curve and the accompanying increase in average, the total percentage of the universe less than the minimum specification limit is extremely small (Table 5).

The average of the random survey data would be expected to be greater than that for the initial control test data as may be seen in Projects 1 and 3. The reverse is shown in Figure 30 for Project 2. The relative positions of the initial-control-tests curve and random survey curve for Project 2 are believed to be the exception rather than the rule. For all three projects, these two curves are generally very similar in appearance considering the different conditions reflected by the two sets of data. Comparison of numerical values may be made by referring to Table 6.

Based on the data shown in Figures 29 through 31, the following comments regarding the effects of representing the universe by the three different sets of data may be made. The final job control data tend to produce an average greater than the true average, assuming that the true average is closely represented by the random survey. The skewed distribution of the final control data results in a misrepresentation of the dispersion of the true universe. For the projects included in this study, the random survey data distributions are believed to closely approximate the true distributions.

As explained in the preceding section, much of the bias in the control test results is due to the practice of resampling. Bias may also be introduced by the long-established practice of selecting samples by nonrandom methods. The control test samples may be selected from only those areas appearing to the sampler to be well compacted or only those areas which do not appear to be well compacted. In any case, it is well established that nonrandom sampling tends to introduce bias (5, 6).

COMPACTION SPECIFICATIONS

The Present Specification Problem

This study illustrates the need for improving present procedures for evaluating embankment compaction. Although current field control procedures may produce results that appear to be compatible with present specifications, the random survey of this study on each project revealed a percentage below the specification requirement. This discrepancy between field control data and statistical estimates has been more of a concern to non-engineering people interested in highway construction than it has to the highway engineer. Even though this difference has not always been measured, the engineer has always been aware of it and considered it within the realm of engineering judgment.

The problem then is that extensive testing of fills reveals that complete statistical compliance with present specifications cannot be achieved because no provision is made for less than 100 percent compliance. Therefore, if future specification requirements are to be enforced to 100 percent compliance, a new embankment compaction specification will be necessary. It should be one that will continue to assure that the present desirable quality level will be maintained, but with which compliance can be achieved using present acceptable construction procedures.

Review of Statistical Specifications

The California Division of Highways has reviewed two general types of specifications which may be adapted to a variety of materials or processes. They are: (a) the type presented by the Bureau of Public Roads and in further detail in Miller-Warden Associates Technical Report No. 201 (6); and (b) the type presented in the AASHO Road Test Report No. 2 (1) and in further detail in Military Standard 414, "Sampling Procedures and Tables for Inspection by Variables for Percent Defective."

From our review, it appears that the theoretical statistical specifications for on-the-job processing of manufactured materials may lead to higher testing costs with no guarantee of increased quality. Significant changes in the testing and inspection procedures could of course change this situation. From the work done by Weber, it appears that an adoption of the area concept method, similar to that which was employed for the construction control of the actual road test, may be economically feasible providing nuclear testing equipment can be employed (13).

A major portion of highway embankments are made from material taken from the cut areas. Since this is state-owned material, it is generally only possible to accept or reject the compaction work done by the contractor. This means that resampling and rerolling must be considered an accepted part of the construction process. Because neither method mentioned above has a procedure for acceptance after reworking and resampling, they would require considerable alteration before they could be successfully used in embankment control. Since quality will change with each reworking, there probably should be no limit on the number of times an area may be reworked and resampled.

Forms for New Specifications

The U.S. Bureau of Public Roads has provided the state with a five-point guide for statistical specifications (15):

1. A statement as to the desired average value of significant characteristics.
2. A definite acceptance criteria. These criteria will consist of numerical upper and/or lower limits for significant characteristics.
3. A definite number of random samples upon which the decision for acceptance or rejection will be based. The number of samples will be determined by the confidence level required, relative to material outside the tolerances.
4. A statement as to the location or point in the process where acceptance samples will be taken, and the method of sampling and testing.
5. A statement as to what action will be taken if acceptance limits are not met.

While this form provides adequate framework for a specification, it was intended to be of a general nature. When considering compaction control specifically, it is the opinion of these researchers after reviewing the data from this study and existing statistical specifications that the above outline should be modified to read as follows:

1. A statement as to the desired average value of relative compaction. (In the case of uniform material, this average should be based on some prior knowledge of the type of soil to be placed in the embankment. When dealing with extremely heterogenous

materials, it may be necessary to make day-by-day adjustments in the specified average.)

2. A definite acceptance criteria which consists of a numerical lower limit for relative compaction. (This lower limit should be established with the prior knowledge of the type of soil to be placed in the embankment.)

3. A statement defining the maximum and minimum size of the compacted area which may be considered as one lot for acceptance testing. (Present thinking is that this should be a field engineering decision as the areas can vary from a few square feet for the structure backfill to wide expanses of fill area.)

4. A definite number of random samples on which to base the decision to accept or reject the state of compaction. (Areas should be defined before tests are made.)

5. A statement as to the point in the compaction process where acceptance sampling is to be done and the exact method of sampling and testing.

6. A statement as to what action will be taken if acceptance limits are not met.

7. A statement defining a procedure for resampling of reworked areas. (This procedure should compensate for the resampling effect. It is not deemed practical to limit the number of times that an area may be resampled; therefore, some sequential sampling procedure should be considered.)

Since all present compaction measuring methods are subject to wide variation and interpretation when applied to various materials, the incorporation of theoretically correct statistical criteria, such as those listed above, probably cannot be economically justified.

California Division of Highways is presently gaining experience with a compaction control specification entitled "Method of Testing for Relative Compaction of Soils by Nuclear Method." This specification, Test Method No. Calif. T231-B, though not a true statistical specification, does incorporate one item which is found in most statistical specifications. Namely, it specifies that multiple testing shall be done in each area and that acceptance of the area shall be judged on the average of six or more test results. This specification is presently being used on 11 embankment construction projects on an experimental basis. The results of this study at this time look very promising. However, it will be approximately a year before the final evaluation can be made.

CONCLUSIONS

1. The results of this study indicate that it would be extremely difficult to prepare an embankment compaction specification based fully on statistical consideration. This is not surprising since most statistical specifications are intended to aid in making a decision to accept or reject material. In embankment construction, the engineer does not reject the embankment material after it has been judged satisfactory for the intended purpose. He accepts or rejects the state of compaction and, in this case, rejection usually means that the contractor must do additional work on the same material.

2. The variation in the statistical distribution of relative compaction values may be quite large depending on the moisture control, uniformity of compacting effort, the variation in the soils, the susceptibility of the soils to this compaction effort, and other differences. Any statistical specification must take into consideration these potential variations from project to project, particularly the variation in the soils.

3. Finished earthwork on the projects surveyed has been judged satisfactory by present engineering standards and is consistent with present specifications, based on field control requirements which include the effort of resampling. However, based on results from randomly selected samples for this survey, the earthwork quality is inconsistent with a strict 100 percent compliance interpretation of the present specifications. This leads to the conclusion that a revision of present specification requirements is necessary if statistical quality control methods are to be used to enforce construction standards.

4. Results of this study indicate that the adoption of purely statistical specifications for compaction using present testing methods (AASHO T180-C and T181-C or Calif. 216) would require an increase in the amount of testing now performed in California.

However, other research work in progress indicates that by the use of a rapid method of testing, such as nuclear testing equipment, it is practical to use statistical specifitions (13).

5. A procedure which allows retesting only of locations having unsatisfactory compaction test results, regardless of whether additional work is performed prior to the retesting, increases the risk of accepting unsatisfactory work.

6. The accuracy of the present control test procedure, California Test Method 216-F, is sufficient to measure significant variations in the percent relative compaction.

7. The distribution of relative compaction values obtained from this survey is believed to be indicative of the range of compaction currently being accepted. For very uniform, non-variable soil, the result of two adjacent relative compaction determinations can be expected to agree within 3 percent relative compaction 95 percent of the time. For highly variable heterogeneous soil mixtures, the results of two adjacent relative compaction determinations can be expected to agree within 7 percent 95 percent of the time.

8. Depending on specific conditions, a contractor must plan to average 93+ percent relative compaction in order to have substantial compliance with the present specification of "not less than 90 percent by the California Test Method 216."

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the many individuals who participated in the compaction portion of the overall statistical survey program. Special thanks are due to the Committee on Statistical Quality Control. The committee formulated the general statistical survey plan for California and assisted in the administration of the individual field surveys. J. C. Obermuller and T. W. Smith offered many helpful suggestions during the preparation of this report.

The authors also extend their thanks to Robert Iliff, who reviewed the statistical analyses; Robert A. Anderson, who reviewed and improved the readability of the final draft; and to William F. Cowden, who prepared the figures and made many of the calculations. Recognition should also be given to William G. Weber for the development of the test method for determining in-place densities using nuclear testing equipment.

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Appendix

TYPICAL EXAMPLE OF RANDOM SAMPLING FROM AN AREA

A portion of a section of roadway which is 30,000 ft long and 26 ft wide is about ready for the cement-treated base. The inspector has been asked to randomly draw 50 samples in duplicate from the section in order to survey the percent of cement in the base material.

Using the attached table of random numbers, Table A-1, the sampler chooses 50 locations in the following manner. Starting at any point on the table and proceeding up or down, but not skipping any numbers, he reads 50 pairs of numbers. In Column 4, reading down, he finds .732, .721; .153, .508; and so on to the fiftieth pair, .698, .539, which is found about midway down in Column 5.

The first, or A, decimal in each pair is multiplied by the length, 30,000 ft, and the second, or B, decimal is multiplied by the width, 26 ft. Each pair of products establishes a coordinate location in a grid system for taking duplicate samples (See Table A-2).

The sampler then plots the 50 locations (Fig. A-1), and numbers them in the order in which the samples will be taken. Should two locations fall so close together that they both could not be sampled properly, the second one is discarded. Returning to the table of numbers, the next, or fifty-first, pair of random numbers is substituted.

Figure A-2 shows how the samples are numbered for identification. Each duplicate sample will be split into two equal portions before being tested.

TABLE A-1

RANDOM NUMBERS

1		2		3		4		5	
A	B	A	B	A	B	A	B	A	B
.576	.730	.430	.754	.271	.870	.732	.721	.998	.239
.892	.948	.858	.025	.935	.114	.153	.508	.749	.291
.669	.726	.501	.402	.231	.505	.009	.420	.517	.858
.609	.482	.809	.140	.396	.025	.937	.310	.253	.761
.971	.824	.902	.470	.997	.392	.892	.957	.640	.463
.053	.899	.554	.627	.427	.760	.470	.040	.904	.993
.810	.159	.225	.163	.549	.405	.285	.542	.231	.919
.081	.277	.035	.039	.860	.507	.081	.538	.986	.501
.982	.468	.334	.921	.690	.806	.879	.414	.106	.031
.095	.801	.576	.417	.251	.884	.522	.235	.398	.222
.509	.025	.794	.850	.917	.887	.751	.608	.698	.683
.371	.059	.164	.838	.289	.169	.569	.977	.796	.996
.165	.996	.356	.375	.654	.979	.815	.592	.348	.743
.477	.535	.137	.155	.767	.187	.579	.787	.358	.595
.788	.101	.434	.638	.021	.894	.324	.871	.698	.539
.566	.815	.622	.548	.947	.169	.817	.472	.864	.466
.901	.342	.873	.964	.942	.985	.123	.086	.335	.212
.470	.682	.412	.064	.150	.962	.925	.355	.909	.019
.068	.242	.667	.356	.195	.313	.396	.460	.740	.247
.874	.420	.127	.284	.448	.215	.833	.652	.601	.326
.897	.877	.209	.862	.428	.117	.100	.259	.425	.284
.875	.969	.109	.843	.759	.239	.890	.317	.428	.802
.190	.696	.757	.283	.666	.491	.523	.665	.919	.146
.341	.688	.587	.908	.865	.333	.928	.404	.892	.696
.846	.355	.831	.218	.945	.364	.673	.305	.195	.887
.882	.227	.552	.077	.454	.731	.716	.265	.058	.075
.464	.658	.629	.269	.069	.998	.917	.217	.220	.659
.123	.791	.503	.447	.659	.463	.994	.307	.631	.422
.116	.120	.721	.137	.263	.176	.798	.879	.432	.391
.836	.206	.914	.574	.870	.390	.104	.755	.082	.939
.636	.195	.614	.486	.629	.663	.619	.007	.296	.456
.630	.673	.665	.666	.399	.592	.441	.649	.270	.612
.804	.112	.331	.606	.551	.928	.830	.841	.602	.183
.360	.193	.181	.399	.564	.772	.890	.062	.919	.875
.183	.651	.157	.150	.800	.875	.205	.446	.648	.685

TABLE A-2

COMPUTATION OF RANDOM SAMPLE LOCATION COORDINATES

Coordinate Along Roadway Centerline			Coordinate Transverse to Roadway Centerline	
Column A Random Numbers (Top Col.4A Down)	Column B Station to Be Sampled (Col.A x 30,000ft.)	Column C Order of Sampling	Column D Random Numbers (Top Col.4B Down)	Distance Form Left Edge of Roadway (Col.D x 26ft.)
.732	219+60	30	.721	19
.153	45+90	7	.508	13
.009	2+70	1	.420	11
.937	281+10	47	.310	8
.892	267+60	42	.957	25
.470	141+00	18	.040	1
.285	85+50	11	.542	14
.081	24+30	2	.538	14
.879	263+70	39	.414	11
.522	156+60	20	.235	6
.751	225+30	32	.608	16
.569	170+70	22	.977	25
.815	244+50	35	.592	15
.579	173+70	23	.787	20
.324	97+20	12	.871	23
.817	245+10	36	.472	12
.123	36+90	6	.086	2
.925	277+50	45	.355	9
.396	118+80	15	.460	12
.833	249+90	38	.652	17
.100	30+00	3	.259	7
.890	267+00	40	.317	8
.523	156+90	21	.665	17
.928	278+40	46	.404	10
.673	201+90	26	.305	8
.716	214+80	29	.265	7
.917	275+10	44	.217	6
.994	298+20	49	.307	8
.798	239+40	34	.879	23
.104	31+20	4	.755	20
.619	185+70	24	.007	0
.441	132+30	17	.649	17
.830	249+00	37	.841	22
.890	267+00	41	.062	2
.205	61+50	8	.446	12
(Column 5A)			(Column 5B)	
.998	299+40	50	.239	6
.749	224+70	31	.291	8
.517	155+10	19	.858	22
.253	75+90	10	.761	20
.640	192+00	25	.463	12
.904	271+20	43	.003	26
.231	69+30	9	.919	24
.986	295+80	48	.501	13
.106	31+80	5	.031	1
.398	119+40	16	.222	6
.698	209+40	27	.683	18
.796	238+80	33	.996	26
.348	104+40	13	.743	19
.358	107+40	14	.595	16
.698	209+40	28	.539	14

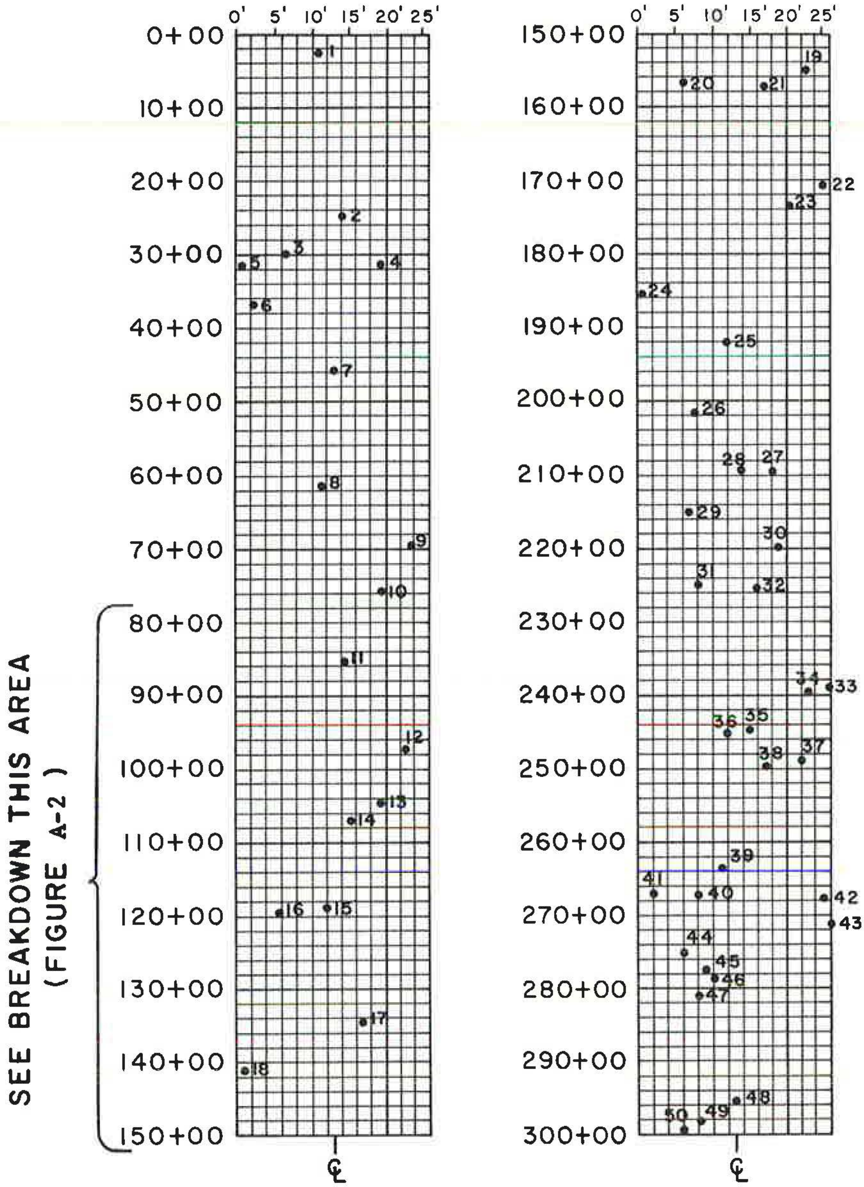

Figure A-1. Typical random sampling from area.

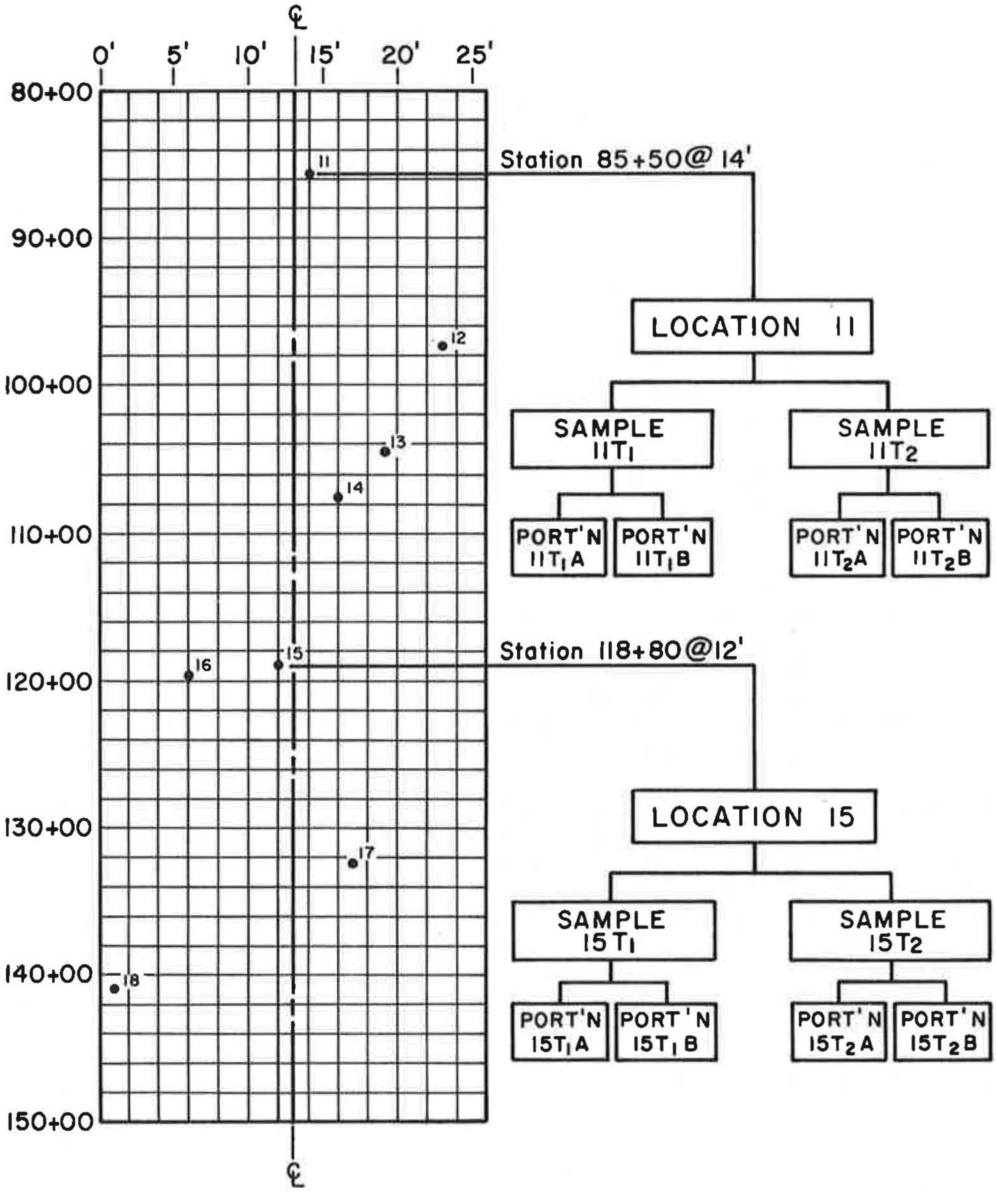

Figure A-2. Typical breakdown of location.

A Critical Review of the Density Testing Program in Washington

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In recent years, Washington has used "end product" specifications for compaction of embankments and has carried out extensive training programs regarding control of compaction and density testing. To review the adequacy of the testing program and to determine the effectiveness of the training programs, a study of density requirements and results was initiated in 1963 and continued in 1964, requiring the review of over 23,000 field density tests.

A computer program based on statistical review of data was utilized to compute and plot curves which assisted in studying and evaluating the testing results for each project and district. A uniformity index was developed and used as a guide for comparing test results and determining what progress had been obtained in the testing program.

The paper describes the improvement throughout the state in density testing and control and summarizes the advantages of the bias testing program over other prepared procedures.

•EVER since the concept was first developed that the strength and stability of earth embankments could be improved by controlling densification through compaction and moisture control, the State of Washington has attempted to utilize this knowledge in highway construction. Early efforts were in the direction of developing construction methods and procedures that would yield adequate results as determined by controlled experiments and tests. This approach proved quite satisfactory until road-building equipment and techniques underwent radical change during and after World War II.

It was soon recognized that specifications based on methods and procedures could not take advantage of economies resulting from the development of new equipment and techniques. In 1952, the Department of Highways started a series of research projects directed at developing compaction control tests and equipment which would permit control of compaction in the field on an "end product" basis. This led to the development of the Washington Densometer, which is a rapid and accurate water-ballon test for determining densities in the field (1), and companion rapid methods for determining the maximum density required, such as a refined one-point Proctor method, and the Washington Method for Determination of Maximum Density for Granular Materials (2).

In 1957, Washington adopted end product specifications for embankments and in 1963 extended this concept to granular surfacing materials. Simply stated, embankments must be built in layers and in accordance with one of three specification methods designated as Method A, Method B, or Method C. All three methods require lift construction, uniform compaction throughout the embankment width and depth, control of moisture content to not more than 3 percent above optimum, and the addition of moisture should it be necessary for proper compaction. The difference between the three methods lies in the thickness of lifts specified, the degree and control of compaction required, and the degree of control of moisture below optimum. The use of suitable

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compaction units is required for Methods B and C, although routing of hauling units may be used to obtain partial compaction.

Method A normally is not specified for state highway work, but may be applied on county or city projects or on certain secondary state highway projects. Embankment lifts up to 2 ft in thickness may be placed, and compaction is achieved by routing the hauling equipment over the entire width of the embankment. It must be determined by inspection that the routing schedule is such that all parts of the fill receive approximately the same amount of compactive effort, including the outer edges of the fill. Drying of soil or addition of moisture may be required if necessary.

Method B is used on all state highway projects, except where other methods are specified. This method requires that the embankment be constructed in lifts not exceeding 8 in. in loose thickness, but lifts in the upper 2 ft should not exceed 4 in. in loose thickness. Ninety percent of maximum density, as determined by ASTM D 698 for fine soils or the Washington Test for granular soils, is required throughout the embankment; however, 95 percent of maximum density is required in the upper 2 ft. Control density tests must be performed to verify compliance with specifications. The contractor is required to dry soil or add moisture as necessary to insure proper, uniform compaction. The selection of compaction equipment or methods is the responsibility of the contractor; however, the use of any method or equipment which does not achieve the required density within a reasonable time may be ordered discontinued.

Method C is required when it is considered essential to the structural quality of the embankment that the entire fill be compacted to a high density, and where the expansive characteristics of the soil dictate a need for a minimum amount of moisture at the time of placement to avoid damaging differential swell after construction. This method differs from Method B in that the entire embankment must be compacted to 95 percent of maximum density, and a limit is specified for minimum moisture content in addition to the maximum. In all other respects the 2 methods are the same, and each requires a high standard of compaction control.

TRAINING PROGRAMS

Prior to 1957, the number of projects requiring Method C compaction and density control tests was relatively small; however, a limited number of personnel had been trained and were utilizing the Washington Densometer on construction projects. After 1957, with the adoption of end product specifications for all embankment construction, it became necessary to train more personnel for inspection and testing of embankment compaction. This was accomplished by placing instructions in the Construction Manual and in yearly training sessions conducted by the headquarters Construction and Materials Divisions. Because of the large increase in the number and size of contracts being constructed, and the increase in the rate of embankment construction brought about by newer equipment and critical scheduling, it soon became necessary to extend the training program to reach even more employees. The yearly construction seminars emphasized the necessity of obtaining uniformity in inspection and compaction of embankments on all projects and explained current problem areas.

In addition, all districts were required to assign an experienced person on a permanent basis as a progress sampler to provide independent checks on the quality of all materials going into the work, and to further provide on-the-job assistance and guidance to field inspectors on proper sampling and testing techniques. The progress sampler works with field personnel and has been of considerable value where circumstances require that inspectors with minimum experience and training must be used.

Another program was initiated early in 1964 wherein a team of 7 experienced employees under the supervision of the construction and materials engineers conducted an inspector training program throughout the state, covering all phases of field inspection. This program involved 5 days of instructions, with one full day devoted to the duties of an inspector on a grading project. The other 4 days included lectures and demonstrations of inspection and sampling procedures for various contract items requiring control sampling, testing and inspection. One session was held in each district, with two sessions being given in the two larger districts. Over 400 state employees attended

WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
DAILY COMPACTION TEST REPORT

Cont. No. 7622
S SH No. 7C
Project Name Delight to Watson Road
Specified Method of Compaction (circle one) A B C

Date 11-25-64
Contractor N. Degerstrom
Res. Engr. D. Walker
Inspector J. Odette

Test No.	Station and Ref. to C/L	Ref. to Grade	Type of Material and Use	Moisture Percent			Percent Passing #4 Sieve	Dry Density pcf					Remarks*
				Opt.	Test	Corr.		Max.	Corr.	Std. No.	Test	% Max.	
33	88+60	-5	Tan Silt	18.9	17.6	—	100	100.6	—	60012	97.0	96.4	
	40' Rt		Emb										
34	99+50	-7	Tan Sandy	16.0	10.1	—	100	105.2	—	60013	101.0	96.0	Rd. Appr.
	100' Lt		Silt -Emb										

Optimum Moisture — Moisture content of #4 minus from Proctor curve.
Corrected Moisture — Optimum moisture corrected for oversize = Optimum Moisture Content × Percent Passing #4 Sieve.
Maximum Dry Density — Density from Proctor curve or Maximum Density for Granular Materials Curve.
Corrected Density — Proctor Maximum Dry Density corrected for oversize.
Standard Number — Laboratory or identifying number of Density Standard used, i.e., Proctor No. or Maximum Density Curve No.
Test — Moisture content or Density of field sample tested.

SUMMARY OF COMPACTION QUANTITIES

Material	Lift Thickness	Compaction Equipment Used (Type and Number of Units)	Number of Coverages Per Lift	Est. Emb. Quant. (or Backfill) Placed (cu. yd.)	Accum. Total Emb. Quant.	Number of Density Tests Taken
Tan Silt	6"	2 Sheepfoot rollers	2	5000	67,000	2
& Sandy Silt						

* If percent of maximum is below specification requirements, note corrective action taken and reference to check test.

Note: If material is gravel and is being placed as any portion of surfacing, indicate the type of surfacing in parenthesis.

Remarks:

R. G.

DISTRIBUTION:
Hdqs. Const. Engr.
District Engr.
Resident Engr.
Dist. Soils Engr.

H. F. 26.73—OS—(Rev. 1-31-64)

Figure 1. Sample of test report.

these sessions. In addition, one session was conducted in Eastern Washington and one in Western Washington for the benefit of the county and city employees. Over 100 county and city employees attended these two sessions.

During the winter of 1965-66, a similar but briefer program was conducted in each district. A list of the items to be covered was sent out by headquarters to the district engineers and the training program was conducted by construction and materials personnel from each district. During these training sessions, inspectors were instructed in the background philosophy of compaction control, the purpose and importance of proper and adequate testing, and the need for uniform inspection and specification enforcement, as well as being given technical training in the control test procedures themselves.

To make a record of tests performed, and to keep the project engineer and others informed as to compaction results, tests and other pertinent data were reported on the Daily Compaction Test Report (Fig. 1). This report provided a basis for evaluating the efficiency of the testing program and revealed areas of training that needed emphasis. For example, during the early stages of its use, retesting of failed areas after correction was seldom being performed. As a result, instructions have been refined and the situation has improved.

Although all concerned have been pleased with the improvement in quality of inspection and control resulting from the intensified training, a need has been felt for some means of measuring the degree of improvement realized. Concepts concerning the adequacy of biased sampling and testing have been set forth by proponents of the statistical approach to process control and product acceptance. To furnish answers to these questions, a study of density requirements and results was initiated in 1963 and continued in 1964, in which all density tests taken on all projects completed during these two years were reviewed. For the 2-yr period 22,300 tests were reported, of which over 7,600 covered Method B compaction.

To clarify factors influencing the data used in these studies, the following inspection criteria are descriptive of the controlling conditions expected.

1. At least one density test should be made for each 2500 cubic yards of embankment placed. Where variable soils occur, more frequent testing should be done. A higher frequency may be necessary in the early stages of construction.
2. Testing should be performed primarily in those areas that appear questionable, with periodic tests being performed in obviously well-compacted areas for confirmation.
3. Maintain a record of compactive effort being applied so that a suitable procedure for compaction can be developed early in the contract.
4. Obtain a standard density curve (ASTM D 698) for each major soil type on the project. Keep a jar sample for quick reference in the field to insure applying the proper standard to the soil being tested.
5. Where soils are being mixed, or where a new soil type is encountered, perform ASTM D 698 on the soil.
6. Report all tests.
7. When test indicates failure to meet specifications, prescribe corrective action and, after the contractor has corrected the area, retest. (Exception: if the original test was within 1 percent of required minimum, retest after correction is not required.)

From the information listed in the compaction reports, a computer program was established which gave the standard deviation, computation and plots of test results for each contract, and summarized these results for each district and for the entire state. Only the results for Method B compaction are included in this report. Figure 2 shows an example of the computer listing, summarizing the compaction results for District 1 during 1963. The left column shows the percent of maximum density and the right column shows the frequency of tests for the corresponding density; the adjacent column lists the percent of the total tests for each corresponding frequency. The listings also show the total number of tests, the mean density and standard deviation. The computer program also gave for each project the cumulative percentage of tests for the respective density, and a plot of the ogive curve for the accumulative percentages (Fig. 2-A). For density tests falling below specifications and when the area received additional compaction and was satisfactorily retested, only the results of the retests

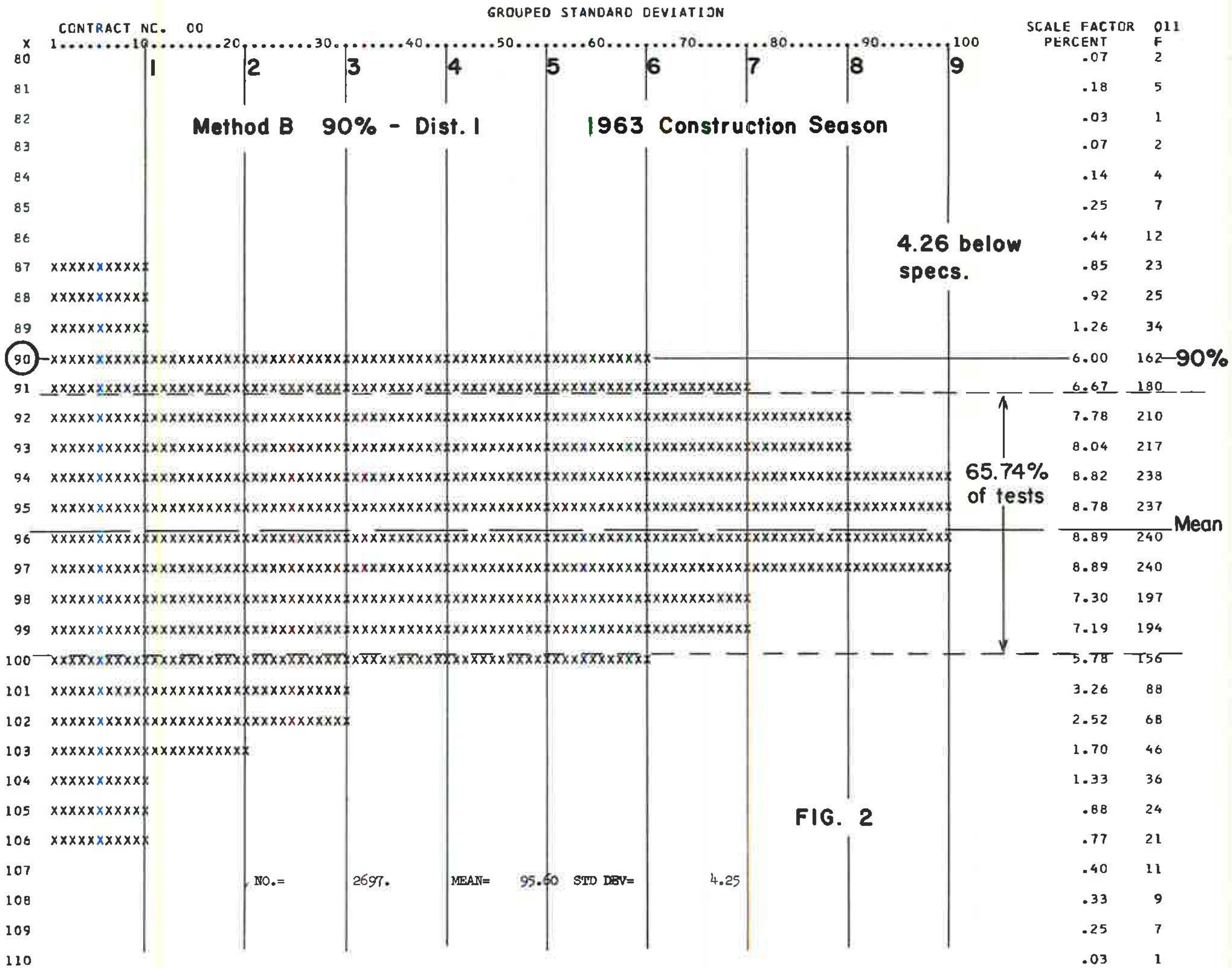

Figure 2. Results of computer program for District 1.

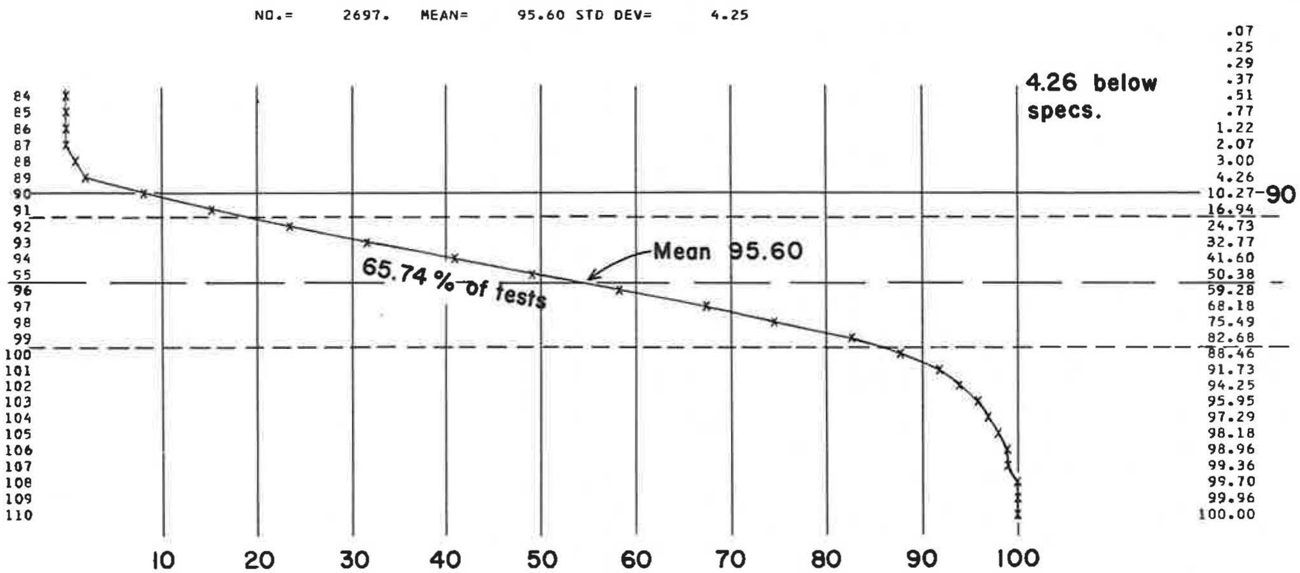

Figure 2-A. Cumulative percentage of tests for each density.

were included in the input data for the computer run. Therefore, the failing tests shown below 90 percent in Figure 2 are those tests for which corrective action was normally indicated, but due to conditions in the field retests were not performed, or if performed were not recorded as a retest.

The number of tests below specification requirements was very small, and on a statewide basis amounted to only 4.0 percent of the total tests for Method B compaction in 1963 and 2.3 percent in 1964. The number of retests performed was approximately 5 percent of the total tests taken. Figure 2 shows 4.26 percent of the tests falling below specifications. In most cases, this is representative of the condition of the embankment at the time tests were taken. Most reports indicate that corrective measures subsequently were employed without retesting, as 65.74 percent of the tests fell within the limits of one standard deviation. This indicates the uniformity of compaction and test procedures. District 1 had the lowest percentage of tests within this range. The sharp rise between 89 percent and 90 percent compaction indicates that the inspectors as a whole were maintaining a rigid control of the work. It appears that tests were taken often enough so that the inspector knew when the specifications were being met. The range of the greatest number of tests (approximately 84 percent) lies between 90 and 100 percent compaction.

About 10 percent of the tests lie above 100 percent compaction. It is possible and feasible to get a density higher than 100 percent by applying more compactive effort at a lower moisture content. Within haul roads and turning areas, compaction is likely to exceed the average density for the embankment and may very well exceed 105 percent in some instances. Out of 39 projects tabulated, 10 projects reported tests higher than 105 percent maximum density, whereas 10 projects did not have any tests over 100 percent. Over half the inspectors on the 39 projects reported densities up to 105 percent. We consider that densities up to 105 percent are valid. They represent 8.5 percent of all tests taken for the 39 projects studied. About 1.5 percent of the tests taken on these projects show densities higher than 105 percent. Although some of these may be valid, it is more likely these are the result of an inspector's error developing from misapplication of the standard density curve.

Table 1 was prepared from the computer listings and gives (for Method B compaction) the comparison of test results for 1963 and 1964. A study of the standard deviation curves and Table 1 shows that with a required density of 90 percent, the curve distribution is on the higher side. Discarding the failing tests and utilization of the retests may account for a portion of this; however, subtracting one standard deviation from the

TABLE 1

COMPARISON OF DENSITY TEST RESULTS FOR 1963 AND 1964
(Method B Compaction)

District	Mean Density		Std. Deviation		Tests Within Std. Dev. (%)		Total Number of Tests	
	1963	1964	1963	1964	1963	1964	1963	1964
State	95.75	95.77	4.34	3.27	71.04	68.44	3798	3881
1	95.60	95.68	4.25	4.18	65.74	67.57	2697	2692
2	97.93	99.16	5.90	5.45	65.83	67.80	125	59
3	95.71	97.25	4.14	4.23	73.90	66.02	249	412
4	96.13	94.67	4.21	3.53	68.44	63.22	215	87
5	96.13	93.92	4.50	4.02	74.73	71.80	182	39
6	95.75	95.11	4.34	3.24	69.40	73.81	330	592

mean density still gives a figure above the specified density in all cases except District 5. It was reported that during 1964, District 5 had only four contracts requiring a minor amount of earthwork with Method B compaction. Most of the earthwork in this District is solid rock for which density tests are not taken. On the four projects involving earth embankments, only 39 density tests were taken of which four failed and corrective action was taken without retesting.

From our own experience and observation and from HRB Bulletin 270, we know that the number of roller passes required to increase the density of the soil from 90 percent to 95 percent is small compared to the number of passes required to increase the density from 95 percent to 100 percent. Normally, Method B compaction test results are well above the 90 percent minimum requirement; however, we are concerned with the high mean densities, especially where the high mean is caused by extremely high densities brought about by excessive rolling. This is an unnecessary expense which is ultimately reflected in high bid prices. In Washington, embankment compaction is paid for by the cubic yard.

A great spread on density results may be caused by many factors, such as existing materials, weather conditions, moisture, compaction effort by the contractor, additional compaction from hauling equipment and misapplication of density standards. Our goal is to achieve uniformity in density testing and enforcement of requirements which should result in reducing extremely low and extremely high density tests. We further

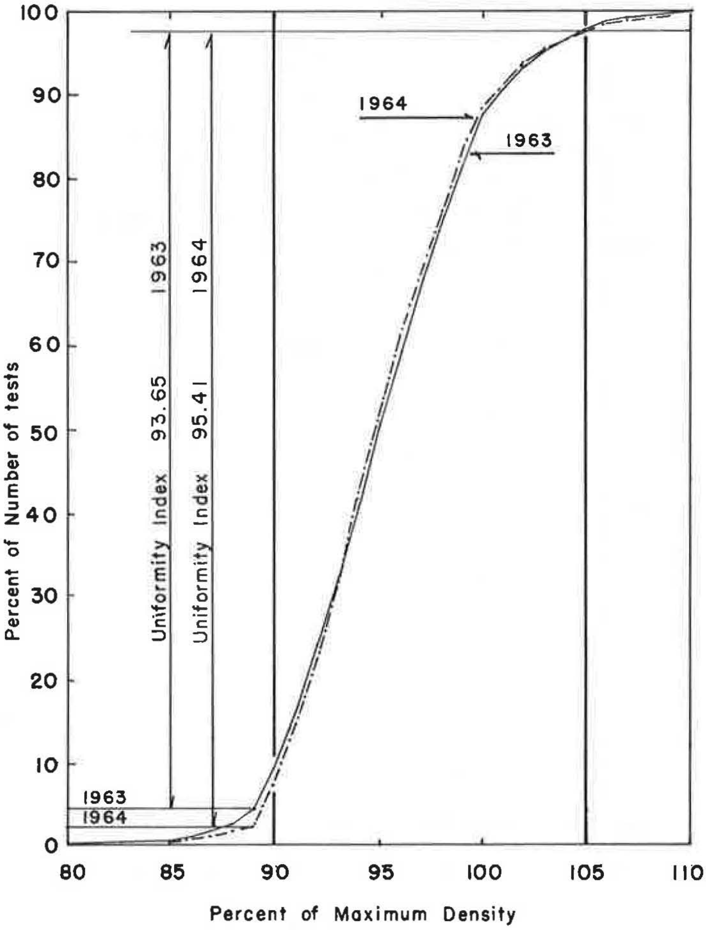

Figure 3. Uniformity index for statewide density testing for 1963 and 1964.

assume that improvement would be shown by a decrease in the mean density to a few percent above minimum requirements, a decrease in the standard deviation, and an increase in the percent of tests within one standard deviation.

It is possible to use each of these factors for comparing improvement on a project or district basis; however, since each factor is influenced by the others, overall comparisons are more difficult.

For example, on a statewide basis in 1964 the mean density stayed about the same, the standard deviation decreased, but so did the percent of tests within the standard deviation (Table 1). From our review, we know that overall improvement did occur, but the data do not clearly indicate this. In District 2 for 1964, the standard deviation decreased, but the percent within the standard deviation increased as did the mean density from the high mean of 1963. If the mean density had not increased, we could definitely say that District 2 improved during 1964. Here again, there are too many variables to contend with. After study, it became apparent some factor other than those indicated would be needed for overall comparison.

On the basis that improvement should be shown by more adequate retesting (which would result in less failing tests being used in the final accumulated data) and fewer densities above 105 percent (which would indicate less misapplication of density standards and less wasted compactive effort), it was concluded that uniformity of test results would be an appropriate indicator for comparison.

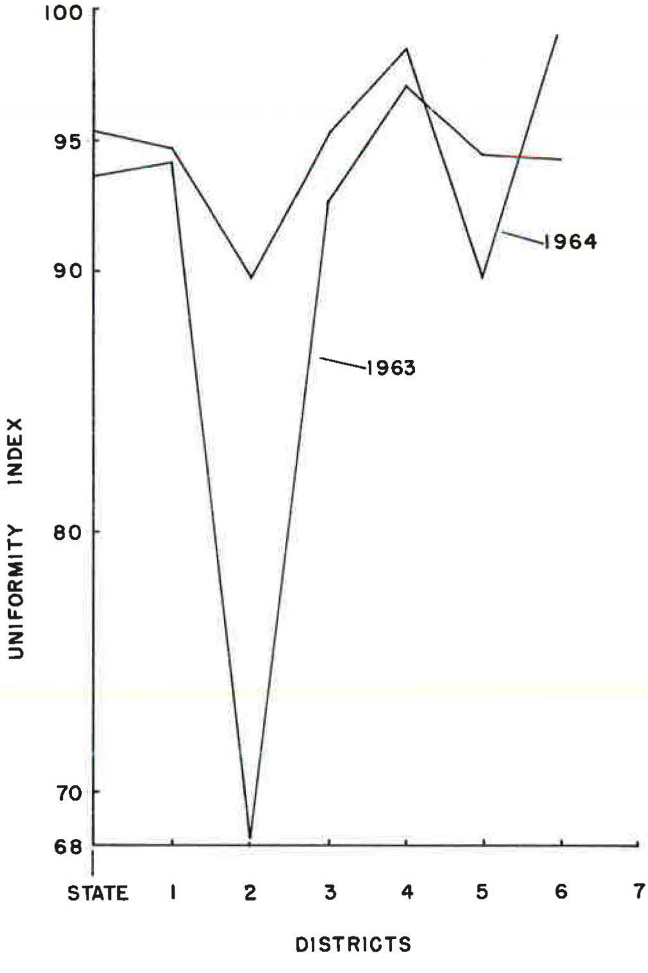

Figure 4. Plot of the uniformity index for all districts for 1963 and 1964.

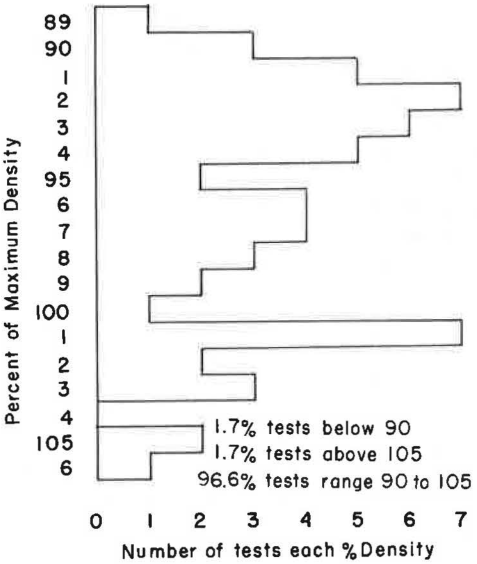

Figure 5. Apparent misapplication of maximum density curves.

From the computer results, the ogive curves were studied and it was noted that the percent of tests falling within specified limits were readily available and could be used for comparison purposes. From the test data for Method B compaction, it was determined that the lower limit for comparison purposes should be the specified requirement of 90 percent density and the upper limit of 105 percent density. A new term called "uniformity index" was coined which, for Method B compaction, is the percent of the number of tests falling within densities of 90 percent to 105 percent. The difference in the uniformity index from 1963 to 1964 (Fig. 3) represents an increase of 1.76 percent of the tests falling within the selected limits. This indicates a very definite improvement.

The uniformity index of each district and statewide for 1963 and 1964 is shown in Figure 4. Improvement is noted in all districts except District 5. As previously noted (Table 1), District 5 had a small number of tests in 1964, with a high percent of failing tests on a minor quantity of embankment. Figure 4 shows where improvement has been the greatest and where additional training is needed.

District 2 shows radical improvement from 1963 to 1964. A review of individual test reports for District 2 shows that the low uniformity index for 1963 is the result of a large number of low tests and extremely high tests. This also accounts for their high mean density and standard deviation given in Table 1 for 1963. The greater emphasis on the use of proper standards is largely responsible for the improvement.

As previously mentioned, the instructions require that a sample of the soil be retained in a sealed jar and identified with the corresponding maximum density curve (ASTM D 698) for each type of soil on the project. This is necessary in order that the proper maximum density curve be used with the corresponding field density test to calculate the percent of maximum density. If the wrong standard curve is used for a given field density test, either high or low results may be indicated and the test has no real value. Present instructions require that the standard density curve be prepared either by the district or the headquarters Materials Laboratory on preliminary samples, and be identified for each project with a standard curve number. Supplemental tests are made in the field as necessary. In some cases where visual identification is difficult, a one-point Proctor test is run for each density test. Based on our review, it appears that misapplication of standard curves is one of the major problems in the density testing program; additional emphasis will be directed toward improving this condition in the future. Figure 5 shows an apparent misapplication of standard curves where two high peaks developed within a reasonable range of densities. Between 89 percent and 100 percent density, a peak occurs at 92 percent; between 100 percent and 196 percent density, a peak occurs at 101 percent. The double peak indicates that the inspector probably used the wrong standard density curve on some tests. There are other factors which could have caused this, such as variability in moisture and compactive effort; however, they would not tend to create two distinct peaks.

SUMMARY

Although studies presented here cover only a 2-yr period, which is admittedly not long enough to form definite conclusions, it is considered that the results warrant extension of the study program on a continuing basis. Consequently, the Office for Construction plans to continue making similar annual studies and to use the uniformity index as a guide to planning inspector training programs and evaluating the effectiveness of the density control program.

The emphasis being placed on good density control procedures and the realization on the part of inspectors that their work is being used, reviewed, and evaluated has created a pride in work and a recognition of importance that did not always exist before. The response to the training program has indicated the need to utilize the intelligence and individual abilities of inspectors and to foster this utilization by delegating responsibility as well as accountability. Without exception, where inspectors are not cognizant of the importance of their work, quality is low.

We have tentatively concluded that the present density control program is adequate and offers several advantages over other prepared procedures for the following reasons:

1. The frequency of testing appears adequate to give full assurance that specifications are being complied with when applied in accordance with present instructions; i.e., testing is concentrated on suspect areas as are revealed by reaction to hauling units and compactors.
2. The procedure results in early detection of non-specification work which permits correction immediately, thus avoiding costly waste and delays.
3. The amount of testing required and the speed with which individual tests can be made (20-30 min) causes little or no delay to production, and the contractor can proceed with reasonable confidence that he has either met specifications or has corrected deficient areas as they occur, thus avoiding the possibility of major rejections or reconstruction.
4. Early in the contract the testing program is directed toward assisting the contractor in establishing a compaction procedure which will assure compliance yet not result in costly over-compaction. This substantially increases the efficiency of the inspectors as they can concentrate on suspect areas.
5. Utilization of the progress sampler and crosscheck and review of results gives further assurance of adequate inspection.

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Density Control: Its Benefits and Complexities

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A review of compaction principles is presented with emphasis on how these matters affect the strength and volume change characteristics of soils and base materials. The Texas Compaction Ratio is also reviewed and the difficulties encountered are discussed. They include inadequate test procedures and job site problems. It is proposed to develop a test method, using a modification of the Texas gyratory equipment, that will make it possible to identify important moisture-density relationships needed in density control from the material excavated in making in-place density tests. A tentative procedure is proposed for further research.

•NO ONE phase of engineering has intrigued as many minds among planning and construction personnel as the principles of compaction control. All of this is amazing because no two organizations use the same techniques for control testing, and every organization meets with many difficulties in trying to carry out high quality density control. Many cling to density control methods even though they know the basis on which they are working is far from desirable, whether it be testing technique troubles or problems concerning specifications. Obviously there must be advantages to using density control methods, otherwise a large number of the present techniques would have been scrapped years ago.

The benefits must be impressive or else our colleges and universities have done a super selling job to newly graduating engineers. The truth probably is that almost any type of density control method improves the workmanship of roadbeds so greatly that we are willing to accept any control method that will not be too hard to enforce. To avoid trouble with enforcement, some accept such low percent density requirements that strengths may be seriously impaired, that is, if the contractor's equipment can leave it that loose during construction. Figure 1 shows that various compactive efforts produce variation in densities of a sandy soil. Figures 2 and 3 show how shear strength of non-swelling soils is enhanced by increased densification. The lower curve in Figure 2 shows that increased densification of clay soil does not always increase shearing strengths but may instead cause a decrease. Other work (1) shows that increased densification of clay soils causes excessive swell. Thirteen years ago a compaction method was developed that would require high densification of non-swelling soils and far less densification of swelling soils (2). This method, known as the Texas Compaction Ratio (Figs. 4, 5), has been widely used with success by the Texas Highway Department and others. In its simplest terms compaction ratio is the degree to which it is desirable to compact soil materials. The degree or change in density is based largely on the compactibility and volume change characteristics of the material.

Compaction ratio (CR) is expressed by

$$CR = \frac{D_A - D_L}{D_D - D_L} \times 100$$

Paper sponsored by Committee on Compaction and presented at the 46th Annual Meeting.

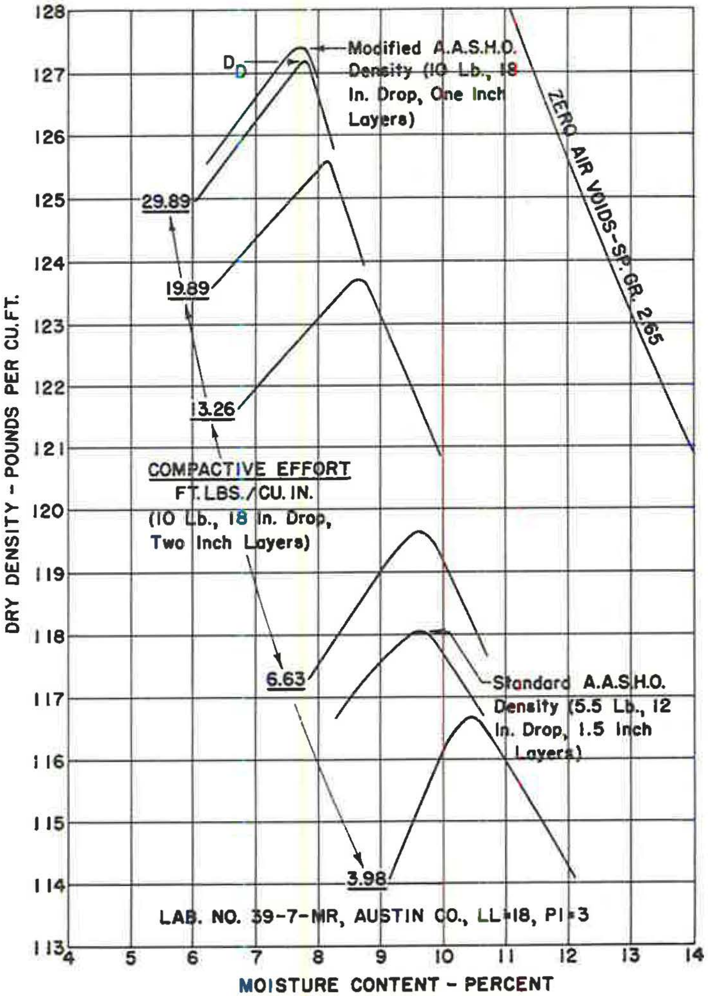

Figure 1. Effect of compactive effort on density.

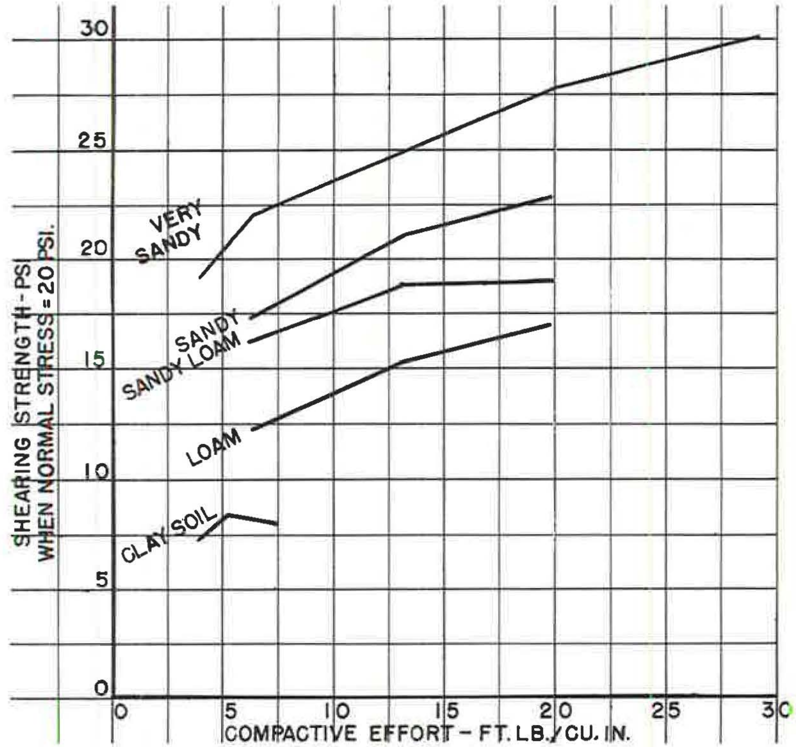

Figure 2. Shearing strength when normal stress = 20 psi vs compactive effort.

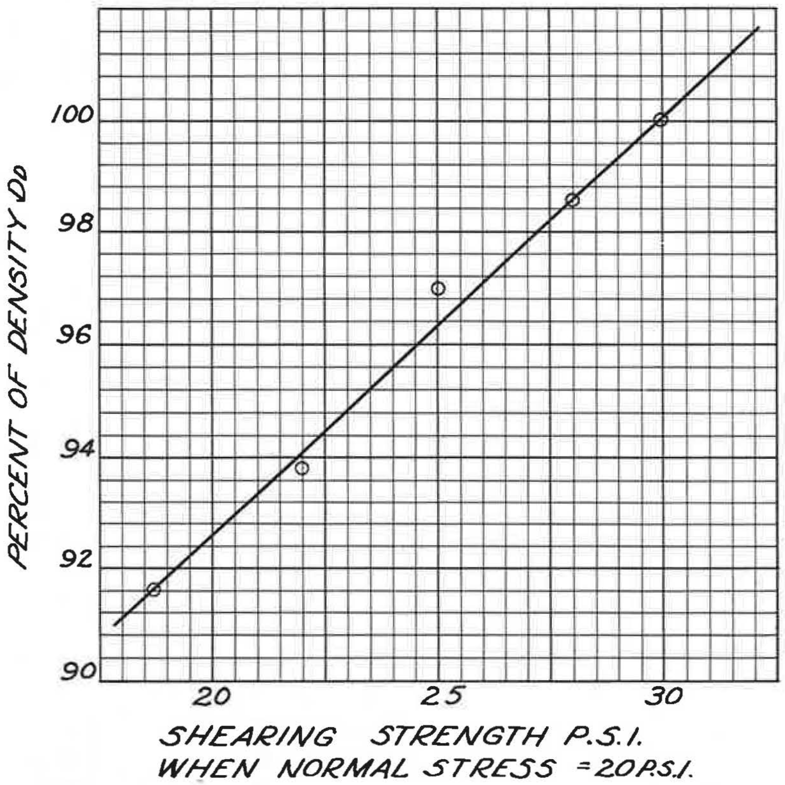

Figure 3. Relation of percent density to shearing strength for sandy soil 39-7-MR.

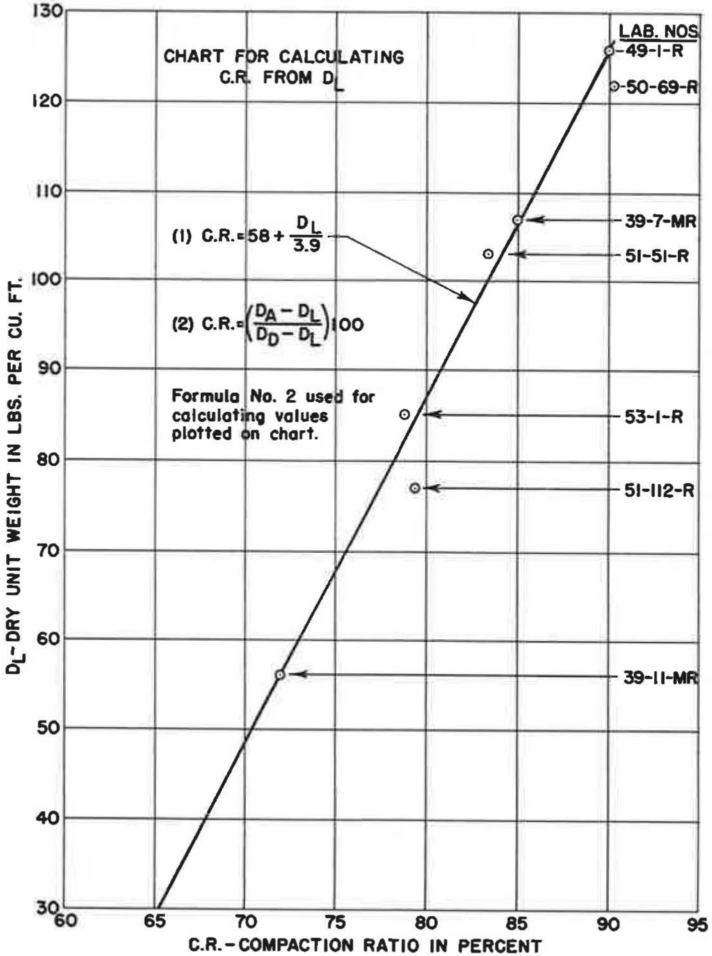

Figure 4. Relation of loose density to compaction ratio.

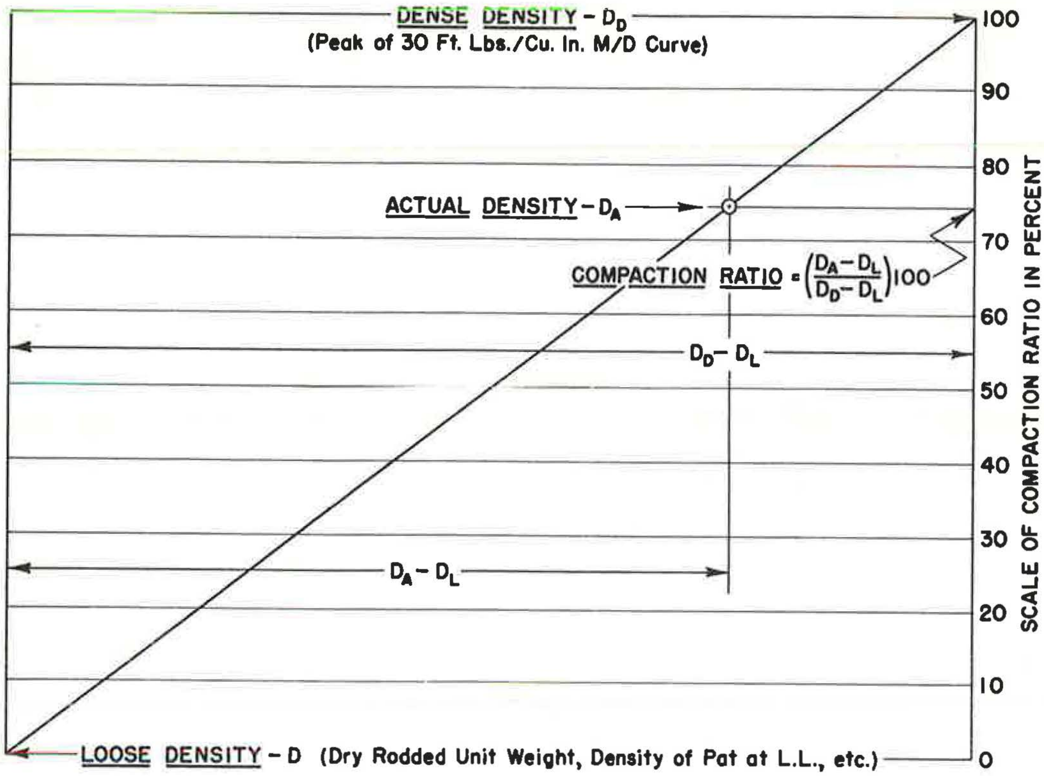

Figure 5. Graphic presentation of compaction ratio.

where D_A is the dry weight per cu ft of material to be obtained in the roadway and D_D is the optimum dry weight per cu ft obtained by running a moisture-density curve using a compactive effort of 30 ft-lb/cu in. (note that D_D in Figure 1 is approximately equal to that obtained by the Modified AASHO Method). D_L for base materials is obtained by use of the standard dry rodded unit weight test; for soils,

$$D_L = \frac{\text{Shrinkage Ratio} \times 62.5}{1 + \frac{\text{LL - Shrinkage Limit}}{100}\,(\text{Shrinkage Ratio})}$$

expressed as unit weight in lb/cu ft.

Figure 6 shows the frailties of percent density. For instance, the lower curve shows that the dry rodded unit weight of a material is nearly 90 percent of that of a very densely compacted material. Figure 7 shows a remarkedly good correlation between compaction ratio density and field densities from well-controlled jobs. Note that the traffic on the WASHO Road Test increased the density up to CR density. Improvement of testing techniques and consideration of problems encountered in the field should enhance the use of density control. As for testing techniques, we have tried to overcome some of the problems common in procedures by introducing the CR method. Among these were some techniques which have been previously recommended (3), such as use of large size material, individual specimens and the 10-lb rammer with 18-in. fall.

We have used high compactive effort tests rather than the standard tests because we believe the results are more nearly reproducible than are the low compactive effort test results. For example, the data in Figure 8 show that the standard compaction test produces moisture-density curves which are separated by a 2 lb/cu ft interval merely by the difference in the level at which the rammer guide is held. The two upper curves

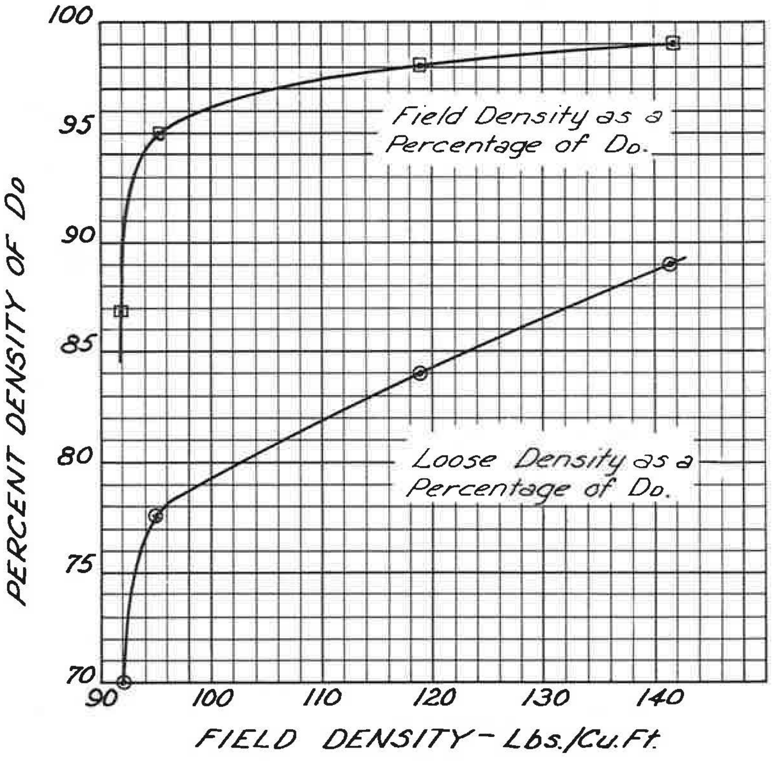

Figure 6. Relation of field density to percent of laboratory densities.

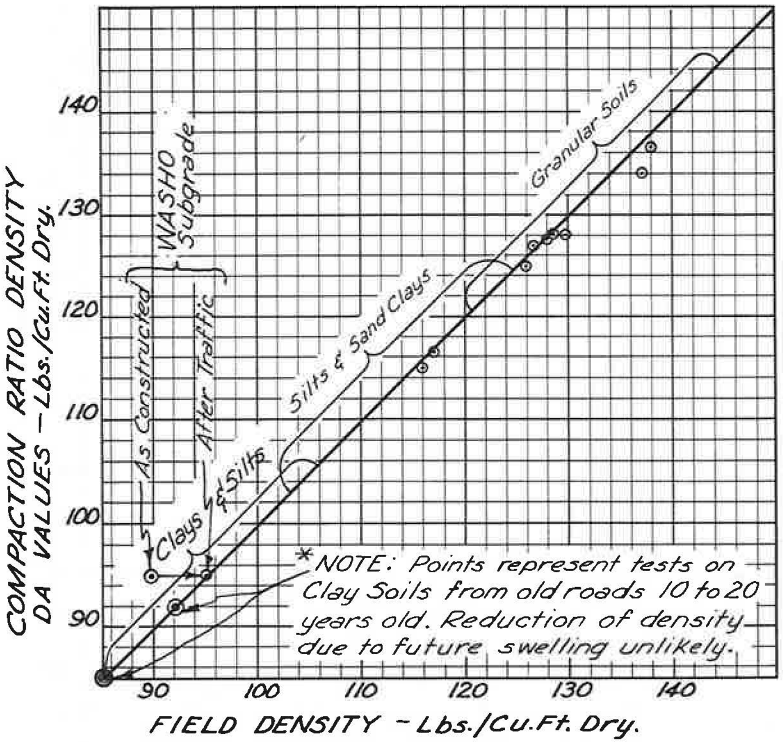

Figure 7. Relation of compaction ratio density to in-place density of subgrade and base materials taken from well-compacted roads.

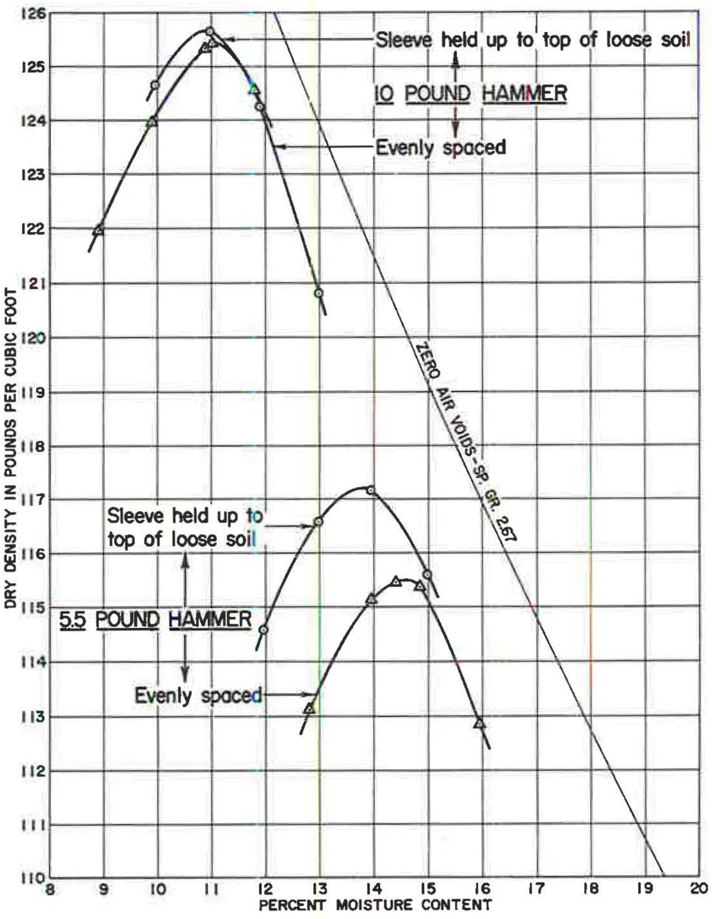

Figure 8. Effect of guide-sleeve level of hand methods on density.

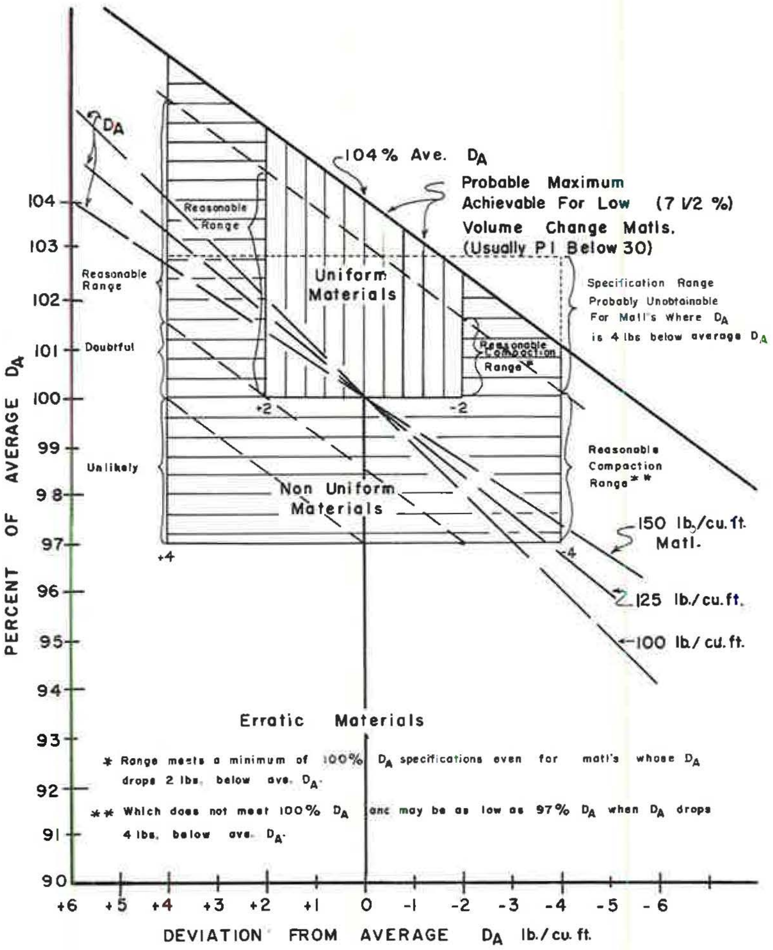

Figure 9. Relation of deviations in density to percent density.

TABLE 1

DESIGN AND CONSTRUCTION SUGGESTIONS BASED ON OBSERVATIONS OF DENSITY CONTROL PROBLEMS IN THE TEXAS HIGHWAY DEPARTMENT

Type of Material	Purpose of Material In Roadway Structures	Recommendations: Percent Average D_A Density	Recommendations: Suggested Design and Construction Practice
Uniform** Subgrade Soils	Upper subgrade strata (6 to 12 In. of subgrade or upper 2 ft. of some subgrades*)	98 to 102% of D_A	Design Pavement for Uniform subgrade support
Uniform** Subgrade Soils	Fills (Lower subgrade strata)	96 to 104% of D_A	Normal Design
Non Uniform Subgrade Soils	Upper subgrade strata (6 to 12 In. of subgrade or upper 2 ft. for some subgrades*)	96 to 104% of D_A	Design Pavement Depth for weaker of soils encountered
Non Uniform Subgrade Soils	Fills (Lower subgrade strata)	96 to 104% of D_A	Normal Design
Non Uniform Subgrade Soils	Subgrade for shoulder widening	Ordinary Compaction	Stabilize and Proof Roll
Lime Stabilized Subgrade Soil	Treatment of very soft subsoils	Ordinary Compaction	Proof Roll
Lime Stabilized Erratic Materials***	Base or subbase over firm subgrade	Ordinary Compaction	Proof Roll
Uniform & Non Uniform Lime or Cement	Base or subbase over firm subgrade	95% D_A Minimum	If subgrade is likely to be weak during construction consider lime stabilization to provide a working table.
Uniform & Non Uniform Soil-Asphalt Stabilization	Base or subbase on firm subgrade	95% D_D Minimum	
Rocky Subgrade with large sizes	Subgrade with over 10% plus 2 In. rock	Ordinary Compaction	Proof Roll
Uniform Base and subbase Materials**	Base or subbase	100% D_A Minimum	Design surfacing for uniform strength base
Non Uniform Materials	Base or subbase	97% D_A Minimum	Design surfacing for weaker of base materials
Erratic*** Base and Subbase Materials	Base or subbase	Ordinary Compaction	Proof Roll
Stabilized Erratic*** Base and Subbase Materials	Base or subbase	Ordinary Compaction	Proof Roll

* High volume change soils on the primary system (PI above 30 and % soil binder above 50).

** Uniform means that for any one given stockpile or working land D_A will not vary more than 4 lbs. Non Uniform means D_A varies between 4 and 8 lbs. per cu. ft.

*** Usually materials which are not crushed and stockpiled and often contain oversize rock which may also have variable gravities and gradations to the extent that D_A varies 8 lbs. or more per cu. ft. in any working land.

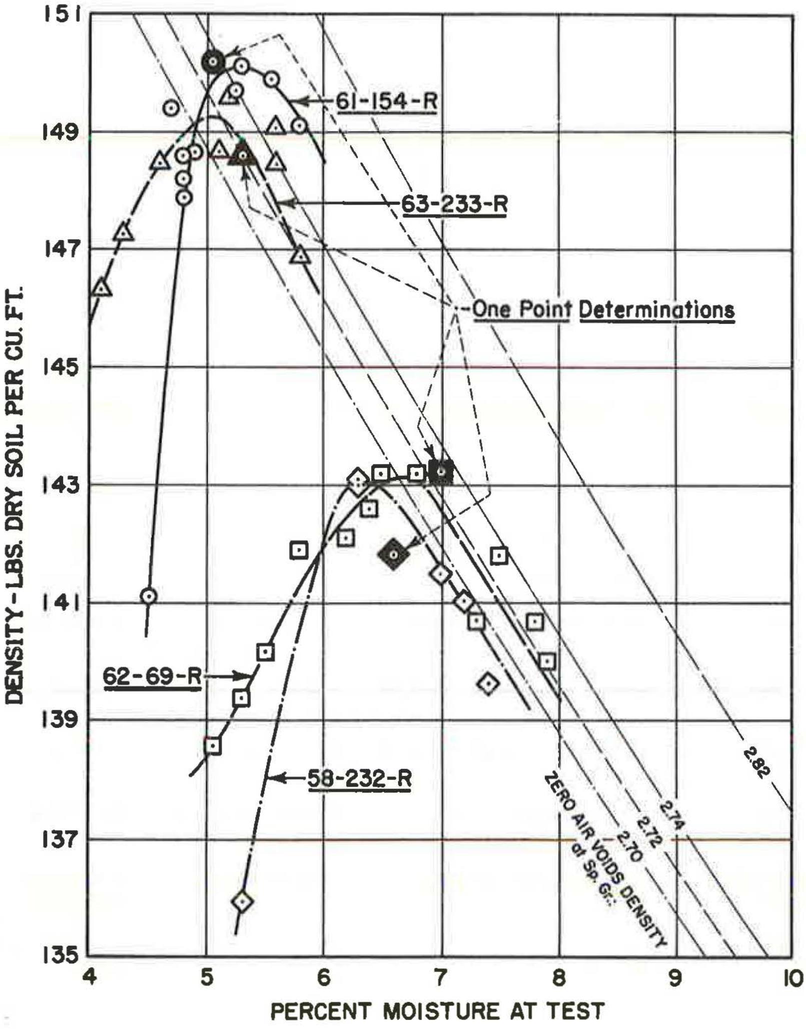

Figure 10. Moisture-density relations obtained by using gyratory equipment (at test).

show that this difference is minimized when the Modified AASHO compactive effort is used. The inefficiency of the standard compactive effort is one reason Proctor did not like the way his test was revised to a 12-in. free fall in lieu of rammer blows applied by arm muscles. If there is any doubt that a difference exists between the standard and Proctor tests, consult the excellent report on compaction by Hveem (4).

We are all guilty of testing unidentified materials for road density and assuming that we have knowledge of the necessary moisture-density relationships for control purposes. It may be that we do have sufficient knowledge for control in the case of testing some uniform materials, but even so, the way a density hole is dug in uniform materials can also have an influence on results.

When erratic materials are encountered, the value of the best techniques and methods often becomes highly questionable. Even when materials are fairly uniform, we are forced to control on the basis of an average "target" density. Figure 9 places all compactible materials into one of three groups, i.e., uniform, nonuniform and erratic, and shows their relation to percent of D_A density. The three long dashed lines intersecting at 100 percent D_A are calculated on the basis of D_A being either 100, 125 or 150 lb/cu ft. Most soil materials fall within these ranges. The upper line on the chart represents what might be called the probable maximum achievable density. This line represents a density somewhat in excess of AASHO modified density for all soil materials except swelling soils. The chart indicates that the uniform materials, i.e., those which have no more than 4 lb deviation in D_A density, can be controlled satisfactorily under present specifications which require a minimum of 100 percent of D_A density. The chart also shows that the lightest of the nonuniform materials probably could not be compacted to 100 percent of average D_A density, and that 97 percent is a more reasonable minimum value. In the case of nonuniform materials, the percent density range based on the average D_A could be anywhere from 97 to 106 percent, whereas each respective material could actually have been compacted to 100 percent of its respective density D_A. Obviously a test for the material excavated from in-place density test holes is needed. Erratic types of material are unlikely to lend themselves to density control specification requirements; however, this does not mean that density tests are of no value in this case. They can be valuable guides in controlling erratic materials, but control specifications should specify ordinary compaction when water and rolling are paid for as separate items.

Until something better is established, it will be necessary to operate on the basis of something similar to the suggestions given in Table 1, in which variations in materials as well as job site conditions and the need of densification are all considered before setting up specifications. In Table 1, the second and fourth types of material involve sublayers which may be either natural soils or layers existing in lower portions of fills. The surcharge load on these layers is such that it would offset swelling pressures sufficiently to permit densities as high as 104 percent of D_A for lower portions of fills, even of the light type. The fifth type of material is involved with a situation of widening an old road where conditions do not lend themselves well to the use of density control specifications. The sixth and seventh types of material may become ample in load-carrying capacities to serve as a working table without having to be compacted to high density. If they have adequate support for construction, there is little doubt of their ability to support traffic after being covered with base and pavement. Greater than normal amounts of rolling on layers above the sixth type of material may cause liquefaction of soft subsoil to the extent that it will squirt out from underneath. If mixtures for the eighth and ninth types of material (Table 1) are properly designed and constructed they should be strong enough to support traffic without requiring high densification. Trying to obtain high densification in these materials by using additional rolling may be detrimental rather than helpful. In general, it may be noted in Table 1 that stabilization of subgrade is recommended. In many cases the primary reason for stabilization is to make it possible to use density control in all layers to be placed. Adoption of the principles and suggested requirements given in Table 1 should lead to the construction of good roads at low cost. Better construction is possible without burdening contractors with the responsibility of compliance when compliance is not possible nor necessary. Better design is also possible because difficulties could usually be recognized soon enough to cope with during the design stage. By using the principles and suggestions proposed, it would be possible to extend the use of density control requirements to many of those who are not using such methods now.

To offer considerable relief from the problems created by attempting to control density of nonuniform and erratic materials, we need to develop a density test which can be run on the material excavated from density holes. We have noted several of the so-called standard moisture-density family of curves and are amazed at the wide divergence obtained by different agencies. One state (5) developed two sets of curves which differ considerably. We have always suspected that it is possible to develop many sets of curves, and for this reason we have not used curves of this type. For this type test to be effective, it would need to be determined from the making of one specimen that is constructed so as to represent or correlate with the density desired. If this could be done it would surely overcome a large part of the troubles encountered in the field, especially when erratic materials are encountered. We know of no better approach to this problem than the use of a modification of our own Texas gyratory test. Figures 10 and 11 show typical M-D curves using gyratory equipment (Fig. 12) to produce specimens approximately 3 in. high and 7 in. in diameter. We plan to mechanize the gyratory equipment, if it continues to be as promising as it now appears to be. Note results of single sample tests. Gradation and soil constants of materials used are given

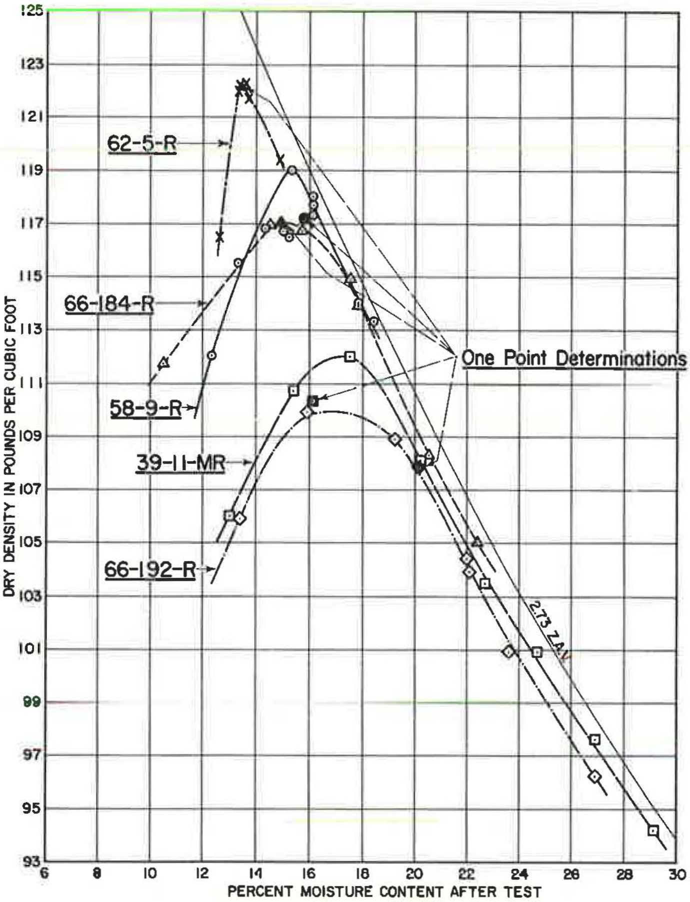

Figure 11. Moisture-density relations obtained by using gyratory equipment (after test).

in Table 2. Figure 13 shows the relation of D_D to be 2.4 lb lower than the gyratory density obtained by using the following procedure:

Preparation of Material

1. Aggregate materials—dry and sieve if sample exceeds 10 lb.
2. Fine grain soils—dry and prepare soil to pass No. 10 sieve.

Molding of Specimen

1. In aggregate materials use a 10-lb sample, wet to the 13.26 ft-lb compaction moisture-density curve optimum, mix as triaxial mix, load in gyratory mold as in triaxial specimen loading, gyrate to refusal at 100, 200 and 300 on press gage and then load to 2410 lb (500 psi) on gage until movement of dial (height) subsides. The moisture-density curve optimum approximates aggregates at saturated surface dry conditions and -40 fraction at PL moisture. In some instances where such information is not available, an abbreviated M-D curve can be obtained by use of the following procedure: (a) compact on dry side of optimum, (b) remove specimen, (c) loosen specimen with ice pick, (d) add additional water, and (e) recompact.

2. In fine grain materials use an amount of soil to make an approximately 3 in. height specimen, wet and mix as a triaxial mix, load and gyrate as mentioned. Load to 2410 lb on press gage and hold until dial is substantially constant. Mixing water is Plastic Limit moisture less 3 percent. In low Plasticity Index materials (less than, say, 15) use mixing water equal to PL -6 percent.

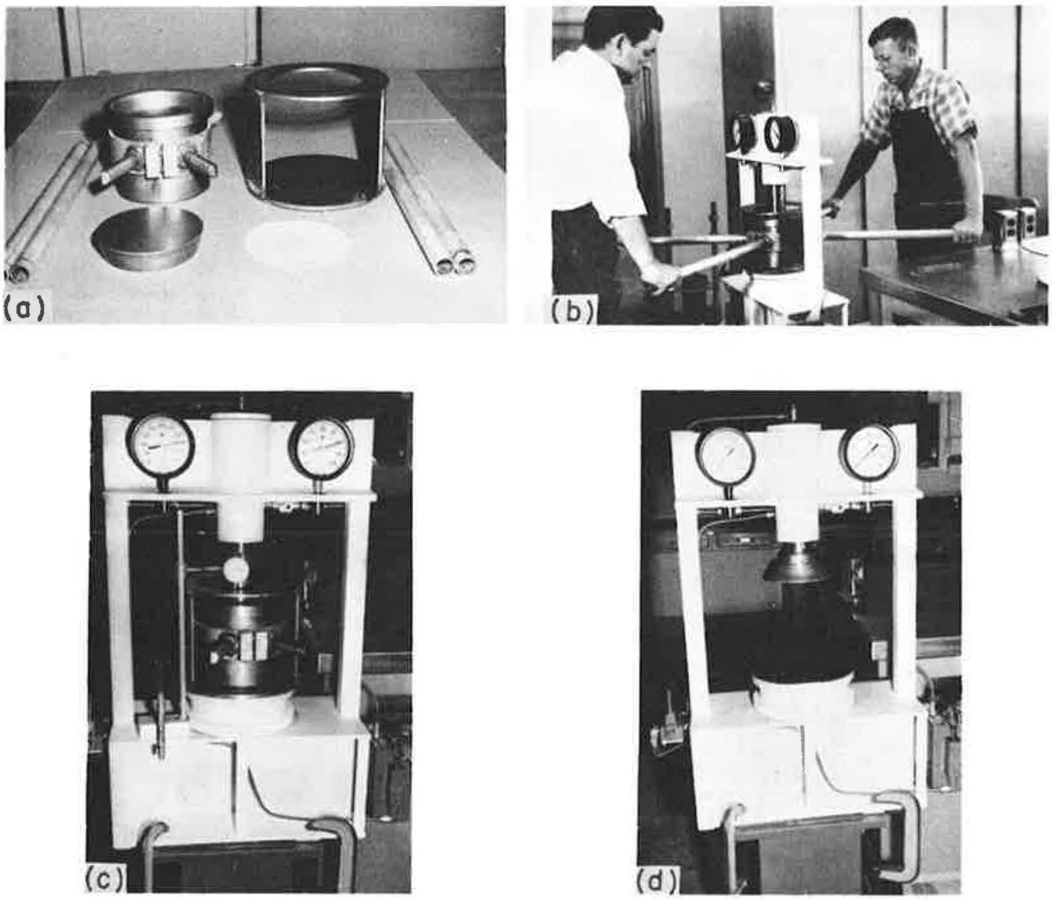

Figure 12. Texas gyratory equipment for base and soil materials: (A) gyratory mold, cage, base plate and handles; (B) equipment in use; (C) measuring loaded height after gyration; and (D) gyratory press showing top plunger head.

3. Additional specimens should be molded 2 percent wetter and drier than those above when material is available.

Then the desired density D_A of the CR Method = (CR/100) (D_G - D_L - 2.4) + D_L. For most base materials except very lightweight materials, it would be reasonable to use a CR value of 89 percent. In this case D_A = 0.89 (D_G - D_L - 2.4) + D_L. Note that only D_G and D_L need to be determined from tests.

CONCLUSIONS

1. Compactive effort affects the degree of compaction which also affects the shearing strength and volume change characteristics of soils.
2. The Texas Compaction Ratio method establishes the densification desired. Such densification is practical and has been obtained under normal job site conditions on a multitude of jobs.
3. Percent density can be misleading and of little value.
4. High compactive effort tests are more reproducible than low compactive effort tests.
5. Calculation of deviations from an average density indicates that density control of erratic materials under the present state of the art is wishful thinking.
6. Job site conditions and the purpose for which the material is to serve play such

TABLE 2

SOIL CONSTANTS AND GRADATION OF GYRATED FLEXIBLE BASES AND SOILS

Constant	Laboratory Number														
	58-232-R	61-154-R	62-69-R	63-233-R	63-282-R	63-383-R	64-413-R	64-459-R	65-67-R	65-100-R	39-11-MR	58-9-R	66-184-R	66-192-R	62-5-R
Liquid limit	29	20	15	18	21	28	32	19	30	24	70	34	42	55	26
Plasticity index	11	6	2	4	7	13	6	5	9	9	41	20	24	32	4
Shrinkage limit	16	13	13	14	14	16	24	14	20	15	11	18	14	16	22
Linear shrinkage	7.2	4.3	1.8	3.3	4.4	6.0	4.0	3.7	5.0	4.7	20.0	8.0	14.0	16.2	3.7
Shrinkage ratio	1.84	1.97	1.85	1.92	1.92	1.88	1.58	1.93	1.68	1.89	1.93	1.75	1.89	1.80	1.70
Soil binder	15	19	47	24	17	34	21	21	29	14	100	94	98	99	100
WBM loss (%)	32	—	—	28	33	45	34	32	37	19	—	—	—	—	—
Percent retained on:															
Square mesh sieves															
1³/₄ in.	0	0	—	0	0	0	0	0	0	0	—	—	—	—	—
1¹/₄ in.	21	3	—	4	9	8	7	6	5	10	—	—	—	—	—
⁷/₈ in.	45	22	—	20	28	21	24	20	18	19	—	—	—	—	—
⁵/₈ in.	56	30	0	30	40	32	35	33	29	31	—	—	—	—	—
³/₈ in.	64	33	5	38	57	42	45	46	42	46	—	—	—	—	—
No. 4	73	45	33	53	70	54	56	59	54	62	—	—	—	—	—
No. 10	80	56	49	64	76	58	66	68	63	73	—	—	—	—	—
No. 20	83	73	50	71	80	61	75	75	68	82	—	—	0	0	—
No. 40	85	81	53	76	83	66	79	79	71	86	0	0	2	1	—
No. 60	86	84	59	81	85	69	82	81	73	88	1	7	5	1	0
No. 100	87	85	69	86	87	72	85	83	75	90	4	34	15	2	3
No. 200	88	86	82	89	89	75	89	85	80	92	8	50	28	3	27
Grain diameter (mm)															
0.05	89	87	85	90	90	77	91	87	85	93	9	51	32	4	43
0.005	94	94	95	96	93	87	97	95	94	97	45	58	55	30	84
0.001	99	97	96	98	98	94	98	98	96	99	59	64	69	62	90
Specific gravity	2.70	2.71	2.59	2.70	2.70	2.73	2.62	2.72	2.67	2.76	2.71	2.69	2.73	2.66	2.67
Material	Flexible base	Flexible base	Subbase	Flexible base	Flexible base	Flexible base	Flexible base	Flexible base	Flexible base	Flexible base	Subgrade soil	Subgrade soil	Subgrade soil	Subgrade soil	Subgrade soil

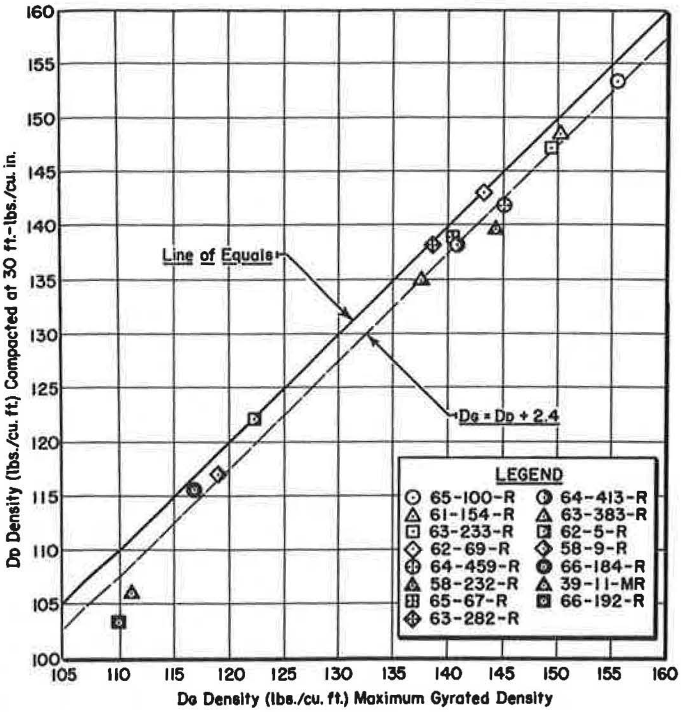

Figure 13. Relationship of maximum gyrated density to D_D, maximum density at 30 ft-lb/cu in.

an important role that specifications often need to be written by engineers who are thoroughly familiar with local conditions.

7. Data obtained by use of the modified Texas gyratory compactor offer promise for an accurate means of measuring important density properties. These properties, when used with compaction ratio data, appear to identify the desired density under normal conditions. Additional research is needed before this test can be considered ready for routine use.

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In Highway Research Record Number 177, on page 210, the following sentence should be added at the end of the third paragraph:

"~~The r~~esearch was sponsored by Douglas County, Nebraska, William Green, County Surveyor."

Retention of Density in Loess Subgrades and Soil-Aggregate Base Mixtures

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•THE strength of flexible pavements depends to a large extent on the strength of the subgrade and base, and the strength of these two layer components depends primarily on their density. The density therefore is of prime importance in the performance of flexible pavements. While the importance of density is generally acknowledged, there are engineers who say that the compaction of the subgrades and water-bound bases is a waste of money since eventually they will de-densify and revert to a loose state such as exists in uncompacted soils.

To our knowledge, there is a dearth of published information on the retention of density in compacted subgrades and bases. No doubt information along this line is available and perhaps some will be furnished in the discussion of the paper.

Our principal purpose is to present data on the retention of density in subgrades composed of loessial soils and in bases composed of aggregates and loess binder. However, since moisture is related to density, moisture contents are included in the research and will be reported also. The objective was accomplished by comparing density and moisture content tests obtained during construction on several projects with similar tests made a number of years after construction.

GENERAL DATA

The research included the sampling and testing of subgrades and bases ranging in age from 4 to 18 years. Eleven projects were investigated, each 1 mile in length. Generally, five locations were tested per mile. The projects are representative of about 200 miles of flexible pavement constructed on mail routes since 1948 in Douglas County, Nebraska.

The paving was done by the stage construction method. Initially, a 20-ft pavement was constructed consisting of a 6-in. compacted subgrade, a 4-in. compacted soil-aggregate base and a prime and double armor coat. Also, a 4 by 12-in. curb, the top of which was even with the top of the base, was constructed along each edge of the subgrade and base. Incidentally, the curb was used to provide lateral support and to prevent progressive disintegration at the edges of the base due to tire traffic.

The second stage consisted of the application of a 2-in. asphaltic concrete mat. The length of time elapsing between stages depended on the traffic count, the service behavior of the double armor, and other factors, such as political influence. Table 1 gives the years in which the first and second stages were constructed on the various projects.

Table 1 also contains the location of the projects investigated, the prevailing traffic counts, the condition of the road surface and the environment. The environment in this case refers to the type of terrain and drainage. Note that with the exception of project 2, all the counts are low. The distress noted in some of the projects refers to the breakup and disintegration of the surface. No tests were made on these areas because the distress was manifestly due to the weakness in the basement soils.

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TABLE 1

PROJECT LOCATIONS AND PERTINENT DATA

Project No.	Road and Location	Environment	Surface Condition	Mixed Daily Traffic Count	Date of Construction	
					1st Stage	2nd Stage
1	Road 60 between roads 5 and 1	Hilly, well drained	Good, one distressed area	276	1948	1953
2	Road 33 between roads 38 and 42	Hilly, well drained	Good	3072	1948	1952
3	Road 82 between roads 41 and 45	High ground, ridge, well drained	Good	431	1948	1955
4	Road 5 between roads 16 and 20	Valley, parallels creek, drainage fair	Good, several distressed areas	496	1950	1954
5	Road 16 between roads 5 and 1	North half, valley, fair drainage; South half, hills, good drainage	Good, one distressed area	776	1950	1957
6	Road 41 between roads 56 and 60	Hilly, well drained	Good, several distressed areas	668	1950	1953
7	Road 21 between roads 46 and 42	Hilly, well drained	Good	250	1951	1956
8	Road 15 between roads 9 and 32	Partly hilly; partly valley	Good	188	1953	1956
9	Road 22 between roads 5 and 1	Valley, parallels creek, fair drainage	Good	273	1954	1961
10	Road 80 between roads 25 and 29	Partly hilly; partly valley, good drainage	Good	227	1957	1965
11	Road 44 between roads 29 and 33	Hilly, well drained	Good	625	1962	1962

Data on Subgrades and Base Mixtures

The subgrade soils consist principally of Peoria loess. In its unadulterated state, this soil passes a No. 200 sieve, contains 10 to 20 percent clay and 80 to 90 percent silt, has a liquid limit of 30 to 40 and a plasticity index of 10 to 15, a maximum density of 104 lb/cu ft dry, and an optimum moisture of 18 percent as determined by ASTM D698-65T.

During construction, the upper 6 in. of the subgrades were compacted to at least 100 percent of maximum density. Because the roads had been graveled prior to the construction of the first stage, some gravel was included in most of the test samples. To compensate for the gravel content in computing maximum density, a formula was developed. Experimentally, the formula is $(A \times 0.364) + B =$ maximum density, in which A is the percentage of gravel and B, the maximum density of the unadulterated soil.

The base mixture used on all the projects except 6 and 11 consisted of 60 percent crusher-run limestone, 35 percent sands and gravel and 5 percent loess soil. This mixture had a maximum density of 143 lb/cu ft dry and an optimum moisture of 5.5 percent. The minus 40 material had a liquid limit of 16 to 22 and a plasticity index of 2 to 7.

The base for project 6 consisted of 90 percent sand-gravel and 10 percent loess soil. The mixture had a maximum density of 138 lb/cu ft dry and an optimum moisture of 6

TABLE 2

GRADATION OF BASE MIXTURES

Project	Percent Retained On:							
	1¼ In. Sieve	¾ In. Sieve	½ In. Sieve	⅜ In. Sieve	No. 4 Sieve	No. 10 Sieve	No. 40 Sieve	No. 200 Sieve
6	—	—	0.0	2.0	11.0	35.0	65.0	88.0
11	—	—	0.0	4.0	19.0	55.0	72.0	93.0
All others	0.0	5.0	—	23.0	37.0	52.0	72.0	85.0

TABLE 3

1966 FIELD DATA, SUBGRADE

Project No.	Test No.	Maximum Laboratory Density (lb/cu ft)	Density		Moisture		
			Weight per Cu Ft (lb)	Percent of Max. Lab. Density	Entire Sample (%)	Gravel-Free Soil: Percent of Sample	Gravel-Free Soil: Percent of Optimum
1	1	105.8	97.9	92.6	24.5	26.0	144.0*
	2	104.8	105.3	100.5*	16.8	17.2	95.5*
	4	104.4	96.6	92.6	21.2	21.4	118.9
	5	108.0	97.3	90.0	20.8	23.4	130.0
	6	104.0	97.3	93.4	19.5	19.5	108.3
2	1	104.0	103.3	99.3	17.2	17.2	95.5
	2	104.0	106.5	102.4	11.6	11.6	64.4*
	3	104.0	97.0	93.3*	25.3	25.3	140.5*
	4	110.6	117.0	105.7	12.6	15.6	86.7
	5	104.0	101.5	97.6	22.1	22.1	122.7
3	1	106.6	106.0	99.5	17.4	18.7	103.8
	2	104.8	106.9	102.0	17.4	17.9	99.5
	3	104.0	95.5	91.8*	24.7*	24.7*	137.2*
	4	107.1	106.4	99.3	18.5	20.2	112.2
	5	104.7	103.5	98.9	19.4	19.8	110.0
4	1	111.2	113.1	101.8	13.5	17.0	94.0
	3	105.8	108.9	101.7	16.3	17.2	95.0
	4	120.1	120.1	97.6	8.5	17.7	98.0
	5	117.5	117.5	103.8	10.6	15.1	84.0
5	1	104.8	106.1	101.2	16.3	16.7	92.8
	2	105.3	95.3	90.4	23.0	23.8	132.2*
	3	108.1	105.7	97.8	15.1	17.0	94.4
	4	110.4	109.0	98.6	12.5	15.2	84.4
	5	105.0	101.1	96.3	19.1	19.6	108.8
6	1	107.0	112.7	105.4	15.1	18.5	102.7
	2	107.7	109.4	101.5	17.0	18.9	105.0
	3	105.1	109.5	104.1	19.3	19.9	110.5
	4	116.7	119.3	102.1	12.4	19.1	106.1
	5	107.6	109.3	101.5	17.0	18.9	105.0
7	4	105.0	104.3	99.4	21.2	21.9	121.6
	5	108.6	102.6	99.4	20.5	23.5	130.6
	6	105.8	110.3	104.2*	16.8	17.7	98.4*
	7	104.0	100.6	96.6	22.9	22.9	127.2
	8	104.0	99.5	95.6	22.4	22.4	124.4
8	1	104.0	102.0	98.1	22.4	22.4	124.4
	2	104.0	104.1	100.0	22.4	22.4	124.4
	3	104.0	96.3	92.6	24.2	24.2	134.4
	4	104.0	100.0	96.2	23.0	23.0	127.7
	5	104.0	96.0	92.2	25.5	25.5	141.6
9	2	106.3	106.0	100.0	15.6	16.7	92.8
	3	104.9	104.0	99.2	20.0	20.5	113.8
	4	105.5	109.1	103.3	17.0	17.7	98.4
	5	105.9	107.2	101.3	17.7	18.7	103.8
	6	104.0	93.5	89.9*	24.4	24.4	135.5*
10	2	106.0	99.9	94.2	17.7	18.7	103.9
	3	109.4	106.6	97.5	16.3	19.1	106.1
	5	104.6	102.0	97.5	18.1	18.4	102.1
	6	109.2	111.5	102.0	15.7	18.5	102.7
	7	104.0	102.1	98.2	19.6	19.6	109.9
11	1	104.0	105.4	100.3	18.1	18.1	100.0
	2	104.0	100.0	96.2	22.8	22.8	126.6
	3	110.1	114.8	104.2	14.6	17.5	97.2
	4	104.0	104.0	100.0	21.6	21.6	120.0
	5	104.0	107.4	103.2	18.9	18.9	105.0

*This value not used in computing averages shown in summary, Table 7.

percent. The minus 40 material had a liquid limit of about 16 and a plasticity index of about 4.

Project 11 was included for the purpose of showing the behavior of an asphalt-treated base. The aggregates in the base were sands and gravel whose minus 40 material was nonplastic. The total mixture contained 3 percent bitumen by weight in the form of emulsion, had a maximum density of 135.7 lb/cu ft, aggregate and bitumen, and an optimum moisture content of 5.2 percent. Typical gradations of each of the three types of mixture are given in Table 2.

SAMPLING AND TESTING PROCEDURES

The sampling and testing during construction was done by standard methods as each layer component was finished by the contractors. The procedure used in April 1966 was as follows: The asphalt surface was removed with a 10-in. diamond-core bit. The base was then sampled and tested by the sand method, ASTM D 1556-64, and then the subgrade was tested by the use of a 3 by 6-in. calibrated tube which was driven into the soil.

The moisture content of both the subgrade and base was determined in the laboratory by the oven method. In order to compensate for the gravel on the maximum density as explained earlier, the gravel content was determined on each subgrade sample by a washing process.

Tabulation of the Test Results Obtained in 1966

The results of the subgrade tests are given in Table 3. This table contains weight per cubic foot, maximum density, relative density and moisture content in the entire sample as well as in the gravel-free soil. The moisture content in the gravel-free soil is also expressed as a percentage of optimum.

The test results obtained for the base are shown in Table 4. This table includes weight per cubic foot, percent of relative density, and moisture content expressed as a percentage of the sample and also as a percentage of optimum moisture.

Tabulation of Test Results Obtained Initially

The test data obtained during construction are given in Tables 5 and 6. Table 5 gives the subgrade tests and 6, the base tests. The type of data in these two tables is the same as given in Tables 3 and 4.

As far as the moisture contents are concerned in the initial set of tests, it must be remembered that the moisture content is that existing when the density test was made. The moisture content depends on how much time elapsed between the time the contractor completed his work and the time the inspector made the test. Furthermore, the moisture contents of the subgrade were no doubt lower than reported when the base was added, and the base moistures lower than reported when the armor coats were added. The intital moisture contents are not of any great significance and are included for the purpose of showing what they were at some time during construction.

Summary of Initial and 1966 Tests

A summary of the test results obtained initially and in April 1966 is given in Table 7. The summary may be used to show a comparison of densities and moisture contents. The individual figures, with few exceptions, are the average of five tests. In computing their averages, individual tests which are manifestly out of line were not inlcuded, as indicated by an asterisk in Tables 3 and 4.

DISCUSSION

A comparison of the data obtained in April 1966 with those obtained during construction substantiates the following:

1. As far as the subgrade density tests are concerned, eight of the projects show densities of 97 to 103 percent of maximum density. This compares well with the intital

TABLE 4

1966 FIELD DATA, BASE

Project No.	Test No.	Density: Weight per Cu Ft (lb)	Density: Percent of Max. Lab. Density	Moisture: Percent of Sample	Moisture: Percent of Optimum Moisture
1	1	145.2	101.5	4.7	85
	2	153.3	107.2*	3.6	65
	3	145.0	101.5	3.8	69
	4	149.2	104.3	3.1	56
	5	149.9	104.8	3.8	59
2	1	146.0	102.0	2.8	51
	2	157.0	109.7	2.4	49
	3	144.3	100.8*	4.4	80
	4	153.1	107.0	3.3	60
	5	152.3	106.5	3.1	56
3	1	145.9	102.0	3.3	60
	3	148.6	104.0	3.3	60
	4	143.6	100.5	5.0	91
	5	145.1	101.4	4.4	80
4	1	147.5	103.1	3.7	67
	2	148.2	103.6	3.0	55
	3	148.4	103.7	3.6	65
	4	145.9	102.0	3.5	64
	5	143.1	100.0	3.9	71
5	1	144.0	100.7	4.5	82
	2	145.4	101.7	5.1	93
	3	145.5	101.7	3.4	62
	4	147.7	103.3	3.6	65
	5	141.1	98.7	3.8	69
6	1	145.6	105.5	3.8	63
	2	145.2	105.2	6.1	102
	3	145.6	105.5	4.8	80
	4	136.9	99.3*	4.8	80
	5	141.2	102.6	3.8	63
7	1	139.2	97.4*	4.3	78
	2	147.9	103.4	4.8	87
	3	149.8	104.7	3.7	67
	4	160.6	113.0*	3.7	67
	5	152.9	107.0	4.0	73
9	1	148.0	103.5	5.7	104
	2	142.2	99.5	6.2	113
	3	142.5	99.7	5.7	104
	4	154.2	107.6*	4.7	85
	5	144.0	100.6	5.9	107
10	1	148.8	104.0	3.8	69
	2	143.3	100.2	4.4	80
	3	147.7	103.1	4.0	73
	4	148.7	103.8	3.0	55
	5	144.9	101.2	3.8	69
11	1	143.4	105.5	0.7	13
	2	141.6	104.2	1.3	25
	3	142.8	105.0	0.7	13
	4	142.6	104.9	0.7	13
	5	139.0	102.3	0.7	13

Note: Base samples were not taken on project 8.

*This value not included in the averages shown in summary, Table 7.

TABLE 5

INITIAL DATA, SUBGRADE

Project No.	Test No.	Maximum Laboratory Density (lb/cu ft)	Density: Weight per Cu Ft (lb)	Density: Percent of Max. Lab. Density	Moisture: Entire Sample (%)	Moisture: Gravel-Free Soil: Percent of Sample	Moisture: Gravel-Free Soil: Percent of Optimum
1	1	104.0	103.5	99.6	16.3	16.3	90.0
	2	118.0	118.0	100.0	11.2	18.2	100.0
	3	108.5	108.6	100.0	11.0	14.7	82.0
	5	104.0	104.5	100.5	11.0	11.0	61.0
	6	104.0	106.0	101.8	15.2	15.2	84.0
2	1	119.6	119.6	100.0	7.2	12.6	70.0
	2	107.2	109.7	102.3	14.0	16.6	92.0
	3	118.0	119.2	101.0	11.3	13.2	74.0
	4	104.0	105.1	101.0	17.1	17.1	95.0
	5	104.0	104.4	100.3	16.5	16.5	92.0
3	1	110.5	111.6	101.0	12.1	18.0	100.0
	2	104.0	105.8	101.7	16.7	16.7	93.0
	3	104.0	104.2	100.1	17.6	17.6	98.0
	4	104.0	103.0	99.0	20.0	20.0	111.0
	5	106.0	106.3	100.2	18.3	19.9	111.0
4	1	104.0	106.7	102.6	16.9	17.2	94.0
	2	109.8	113.9	103.6	13.3	16.0	88.0
	3	105.9	115.3	108.9	13.6	5.0	27.5
	4	109.1	112.8	103.5	15.4	13.5	74.3
	5	116.7	119.1	102.1	10.9	35.0	192.6
5	1	112.0	113.4	101.3	12.2	15.7	87.0
	2	104.0	106.5	102.4	17.3	17.3	96.0
	3	107.3	109.6	102.1	18.2	20.0	111.0
	4	105.8	111.0	104.8	15.5	16.3	91.0
	5	109.5	109.7	100.1	14.3	16.8	93.0
6	1	104.0	104.1	100.0	18.8	18.8	104.0
	2	109.0	108.0	99.1	16.1	18.7	104.0
	3	104.0	105.0	100.8	15.3	15.3	85.0
	4	106.4	107.5	101.0	13.9	13.9	78.0
	5	104.0	105.0	100.8	16.5	16.5	92.0
7	4	112.0	109.3	97.7	15.7	20.1	112.0
	5	120.1	119.1	99.2	9.9	21.0	114.0
	6	108.0	108.0	100.0	17.6	19.8	110.0
	7	120.5	118.9	98.8	10.5	21.0	114.0
	8	118.8	115.4	97.2	11.9	20.2	112.0
8	1	104.0	105.5	101.4	16.8	16.8	94.0
	2	111.2	112.2	100.8	17.8	17.8	99.0
	3	111.2	110.0	99.7	16.7	16.7	93.0
	4	104.0	105.0	101.0	17.5	17.5	98.0
	5	104.0	105.0	101.0	16.5	16.5	92.0
9	2	113.1	114.4	101.0	16.3	20.9	116.0
	3	108.0	108.0	100.0	15.4	16.7	93.0
	4	108.7	109.2	100.5	16.7	18.5	103.0
	5	109.0	109.8	101.0	19.5	21.9	122.0
	6	109.0	109.2	100.0	16.8	19.9	110.0
10	2	117.5	117.6	100.0	9.7	15.5	86.0
	3	129.1	129.1	100.0	5.7	18.4	102.0
	5	133.5	134.0	100.3	5.3	19.5	108.0
	6	119.0	125.0	105.0	7.5	17.9	100.0
	7	114.9	115.2	100.2	10.6	15.4	86.0
11	1	111.6	118.5	106.0	13.7	17.3	96.0
	2	131.8	135.6	103.0	4.5	18.7	104.0
	3	122.6	128.5	104.7	8.3	16.5	92.0
	4	115.9	116.1	100.2	10.8	15.9	89.0
	5	118.0	119.1	100.8	6.8	11.0	61.0

TABLE 6

INITIAL DATA, BASE

Project No.	Test No.	Density: Weight per Cu Ft (lb)	Density: Percent of Max. Lab. Density	Moisture: Percent of Sample	Moisture: Percent of Optimum Moisture
1	1	148.0	103.4	2.6	47
	2	141.6	99.1	2.1	38
	3	144.2	100.9	3.0	55
2	1	142.3	99.6	2.1	38
	2	148.0	103.4	2.6	47
	3	144.1	100.8	2.0	36
	4	143.5	100.3	2.0	36
	5	144.1	100.8	2.1	38
3	1	142.8	99.9	3.6	65
	2	144.9	101.2	2.9	53
	3	142.8	99.9	3.2	58
	4	144.3	101.0	3.2	58
	5	141.3	98.8	3.1	56
4	1	144.0	100.6	3.7	59
	2	150.5	105.2	3.0	55
	3	145.2	101.7	3.6	65
	4	143.0	100.0	3.5	64
	5	143.3	100.2	3.9	71
5	1	144.3	101.0	3.0	55
	2	143.8	100.5	2.6	47
	3	143.0	100.0	2.8	51
	4	147.5	103.2	2.7	49
	5	143.0	100.0	3.1	56
6	1	137.3	99.3	3.0	50
	2	135.8	98.2	3.6	60
	3	136.0	98.6	2.8	47
	4	137.9	99.9	3.1	52
	5	137.3	99.3	2.7	45
7	1	143.0	100.0	2.3	42
	2	142.2	99.5	2.5	45
	3	143.9	100.7	2.1	38
	4	147.7	103.3	2.0	36
	5	142.8	99.7	2.6	47
9	1	139.0	97.3	4.9	89
	2	137.1	99.2	4.1	75
	3	143.9	100.7	5.3	96
	4	143.0	100.0	4.7	86
	5	143.7	100.6	5.7	104
10	1	146.5	102.6	2.9	53
	2	149.2	104.3	1.9	35
	3	148.7	104.0	2.5	45
	4	155.1	108.5	2.9	53
	5	148.7	104.0	1.8	33
11	1	135.7	100.0	2.7	49
	2	139.2	102.8	3.3	60
	3	138.5	102.2	2.5	46
	4	138.4	102.1	2.7	49
	5	140.3	103.5	2.5	46

Note: Base samples were not taken on project 8.

TABLE 7

SUMMARY OF INITIAL AND 1966 TESTS

Project No.	Moisture (%)		Density (% of Max. Lab. Density)		Condition of Surface	Traffic Count
	Initial	1966	Initial	1966		
Subgrade						
1	89	119	100	92	Good, one distressed area	276
2	85	102	100	101	Good	3072
3	103	106	100	100	Good	431
4	91	93	103	101	Good, several distressed areas	496
5	92	106	100	97	Good, one distressed area	776
6	93	95	101	103	Good, several distressed areas	668
7	112	126	101	96	Good	250
8	95	124	100	96	Good	188
9	109	102	100	101	Good	273
10	94	105	100	98	Good	227
11	95	106	100	101	Good	625
Base						
1	47	67	101	103		
2	39	54	101	106		
3	58	73	100	102		
4	63	65	101	102		
5	52	74	101	101		
6	51	72	99	104		
7	42	74	101	105		
9	90	86	100	101		
10	44	69	104	102		
11	50	16	102	104		

density. However, in two of the projects, the density now is only 96 percent of maximum and one of the projects shows only 92 percent of maximum density.

2. All the subgrade moisture contents in 1966 are higher than they were initially. However, the moisture content in eight projects does not exceed 106 percent of optimum. In projects 1, 7 and 8, the moisture content ranges from 119 to 126 percent of optimum.

In connection with these observations on moisture contents, although low average moisture contents are shown in projects 2, 3, 4, 9 and 11, each of the projects mentioned contains one test in which the moisture was quite high. Some of the locations are in low filled areas, but some are in hilly areas. We cannot account for the high results in the latter.

3. The 1966 tests on the base show that the moisture content has not reached optimum in any of the projects and that the density has increased in 10 of the 11 projects. The average moisture content is 70 percent of optimum and the average density is 103 percent of maximum.

The moisture content of the base in project 11 deserves special mention. Note in the 1966 survey that the mixture on this project, which is an asphalt coated aggregate, is only 16 percent of optimum, whereas it was 50 percent initially. The only plausible explanation we can offer is to assume that this base dried out before the asphalt mat was applied and that it did not absorb any moisture subsequently.

4. There appears to be no relationship between age and retention of density in the subgrade. For instance, projects 7 and 8, constructed in 1951 and 1953 respectively, have lower densities than projects 1 and 2, constructed in 1948.

5. There appears to be no relationship between subgrade density and traffic count. This is no doubt due to the fact that with one exception the traffic counts are low. Project 2, with a traffic count of about 3,000, is in excellent condition. This indicates that 4 in. of base and 2 in. of asphaltic concrete can handle considerable traffic.

6. There appears to be no relationship between density in the subgrade and surface condition. The surface condition is good on all projects. The distress which exists on projects 4 and 6 is manifestly due to improper fills or inherently weak basement soils.

CONCULSIONS

This research seems to warrant two general conclusions. First, compacted soil-aggregate bases retain their initial densities very well even if the underlying subgrade becomes quite wet. Second, compacted loessial soil subgrades in flexible pavements retain densities equal to or in the vicinity of maximum in 91 percent of the projects investigated.

The results of this investigation should also serve to dispel the fears of those who are skeptical about the permanency of compaction of soils and soil-aggregate mixtures.

New Method for Laboratory Soil Compaction by Vibration

LARS FORSSBLAD, Head of Research Department, AB Vibro-Verken, Solna, Sweden

A laboratory compaction method based on the use of a vibrating tamper has been developed at the Concrete and Soil Laboratory of AB Vibro-Verken, Solna, Sweden. The weight and vibration data of the tamper have been selected to obtain with cohesionless soils the same maximum density as obtained with the modified AASHO method. For cohesive soil the vibration method has given somewhat lower values, compared with the modified AASHO method. The vibration method is quicker than the modified AASHO test. The vibrating tamper can also be used to compact test cylinders of soil-cement, asphaltic concrete, etc.

•A LABORATORY compaction method based on vibration has long been considered to have certain advantages. The following reasons may be stated:

1. In cases where the field compaction is performed by vibratory compactors a laboratory test by vibration should give best correlation with field results.
2. A laboratory compaction test based on vibration is more easily adaptable to a test mold of greater diameter than the 4-in. cylinder usually employed for laboratory compaction tests. In a larger mold, the tests can be carried out on samples containing material of a larger maximum size.
3. A laboratory compaction test based on vibration is less affected by the manual performance of the test compared with the ordinary Proctor compaction test. A method based on vibration is also quicker and involves less labor.

EARLIER VIBRATION METHODS

Laboratory compaction methods based on vibration of the soil have been tested previously (1, 2, 3). Best known is the Bureau of Reclamation method for determining the relative density of cohesionless free-draining soils (4). In its original version, the sample is vibrated in a container in a saturated state to obtain the maximum density. The Bureau has subsequently issued a modified test method in which a loading weight is placed on top of the material during vibration. The test is performed either on saturated material or on totally dry material. The mold is 6 in. in diameter and 0.1 cu ft in capacity or 11 in. in diameter and 0.5 cu ft in capacity. The method has been approved as ASTM Standard D 2049.

VIBRATING TAMPER TEST

The writer has previously reported (5) comparative tests using three variants of an apparatus for laboratory compaction by vibration: (a) vibration in an open mold fixed on a vibrating table; (b) vibration in a mold fixed on a vibrating table, with a loading weight on top of the material; and (c) vibration with a vibrating tamper working on the top surface of the material.

The use of a vibrating tamper for laboratory testing has been the subject of further studies at the Concrete and Soil Laboratory of AB Vibro-Verken. An advantage of the

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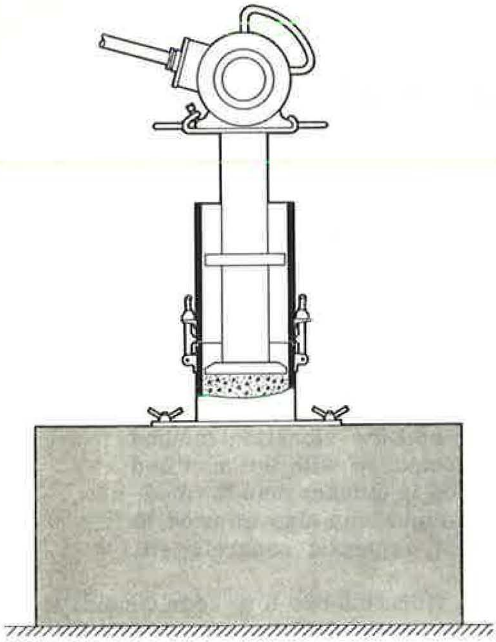

Figure 1. Apparatus comprising test mold with detachable collar and vibrating tamper; the mold is fixed by bolts to a concrete base.

Figure 2. Vibrating tamper, test mold and collar.

vibrating tamper is that a greater pressure is achieved during compaction than by either of the other two methods. The tamper thus gives compaction results that most closely agree with those obtained in the field. The compaction effect is determined by the dimensions of the mold, the layer thickness, the duration of vibration, the weight of the tamper, and the frequency and centrifugal force of the vibrator. These parameters can easily be standardized.

Study Stage 1

The first stage of continued investigations of the new laboratory compaction method concerned an apparatus with specifications as given below and shown in Figures 1 and 2.

(1a) Mold of 1 cu dm (0.035 cu ft) capacity

Diameter of mold 102 mm (4 in.)
Height of mold 123 mm (5 in.)
Weight of vibrating tamper 25 kg (55 lb)
Frequency 3000 vib/min, centrifugal force 225 kg (500 lb)

(1b) Mold of 2.5 cu dm (0.09 cu ft) capacity

Diameter of mold 152 mm (6 in.)
Height of mold 138 mm ($5\frac{1}{2}$ in.)
Weight of vibrating tamper 45 kg (100 lb)
Frequency 3000 vib/min, centrifugal force 400 kg (900 lb)

In a mold with 6-in. diameter the maximum particle size of the soil is 30 to 40 mm ($1\frac{1}{4}$ to $1\frac{1}{2}$ in.). The weights and the vibration data of the tampers have been selected to obtain with cohesionless soils the same maximum density as obtained with the modified AASHO method.

A guiding aim has been to make the new testing procedure as quick and as simple as possible. Filling and vibration in only two layers has been found feasible. The duration of vibration was set at 1 min per layer and the weights and centrifugal forces were selected to give the desired compaction effect under these conditions.

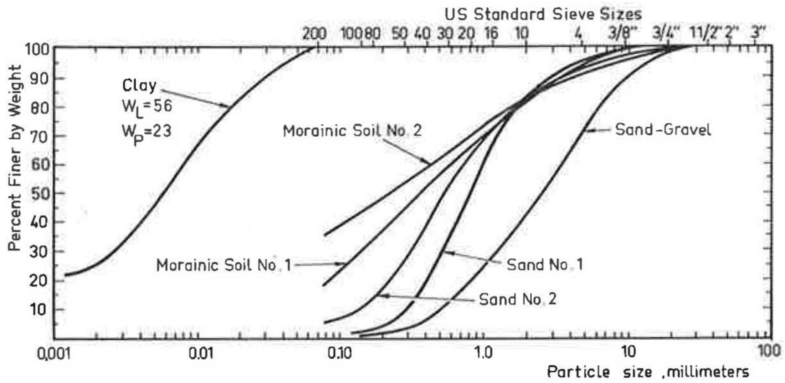

Figure 3. Grading curves for soils used for compaction tests.

The studies have also shown that granular soils can be vibrated with good results in a saturated condition, again filling and vibrating in two layers, each for 1 min. The mold was partly filled with water when the material was placed.

After compaction, and with the surface of the material somewhat above the upper edge of the cylinder, the collar is removed and the material leveled off. As an alternative, it is also possible to compact the material so that its surface is somewhat lower than the upper edge of the mold cylinder, and then measure the distance from this edge. The volume of the compacted material is then calculated on the basis of this measurement.

Grading curves of the soils used for the laboratory compaction tests described are shown in Figure 3. The results of the tests on these materials are shown in Figure 4.

Tests of cohesionless soils have shown good agreement with the results of tests by the modified AASHO method. The highest densities were obtained in vibration tests on dry and saturated materials. Examples of the relationships between the dry density and time of vibration are shown in Figure 5.

Tests of cohesive soils by the new method have given somewhat lower density values than obtained by the modified AASHO method. For clay, the maximum density obtained by vibration was about 10 percent lower than the maximum density obtained by modified AASHO (Fig. 4). In field compaction, it has very often been found impossible to achieve a high degree of compaction for cohesive materials, when compared with the maximum dry density obtained with the modified AASHO procedure. Thus the vibration method in this case gives better agreement with field results.

Practical Application of the Method

The new laboratory compaction method can, of course, be used in the same way as ordinary Proctor tests, that is, by subjecting a given material to a series of tests at various water contents. On many working sites, however, the grading of the soil varies considerably. This means that field density tests must be supplemented by a large number of laboratory compaction tests. Furthermore, the soils often contain gravel or stones in a greater or lesser amount; thus, it is necessary to make the laboratory compaction test with the larger particles removed. Density figures are then corrected with respect to the stone content of the soil, which introduces an element of uncertainty. These problems are eliminated by the following procedure, which is useful for sand, gravel and other granular soils containing a maximum of between 10 and 20 percent of material smaller than 0.074 mm grain size (No. 200 sieve).

Density determinations in the compacted fill are carried out in the ordinary way, e.g., with the water balloon method. At each testing point a soil sample is taken. The sample is vibrated in a saturated state by the laboratory method described previously, and a density value is obtained which in most cases closely agrees with the maximum density obtained by compaction according to the modified AASHO method. This density value is directly comparable with the density value determined in the field.

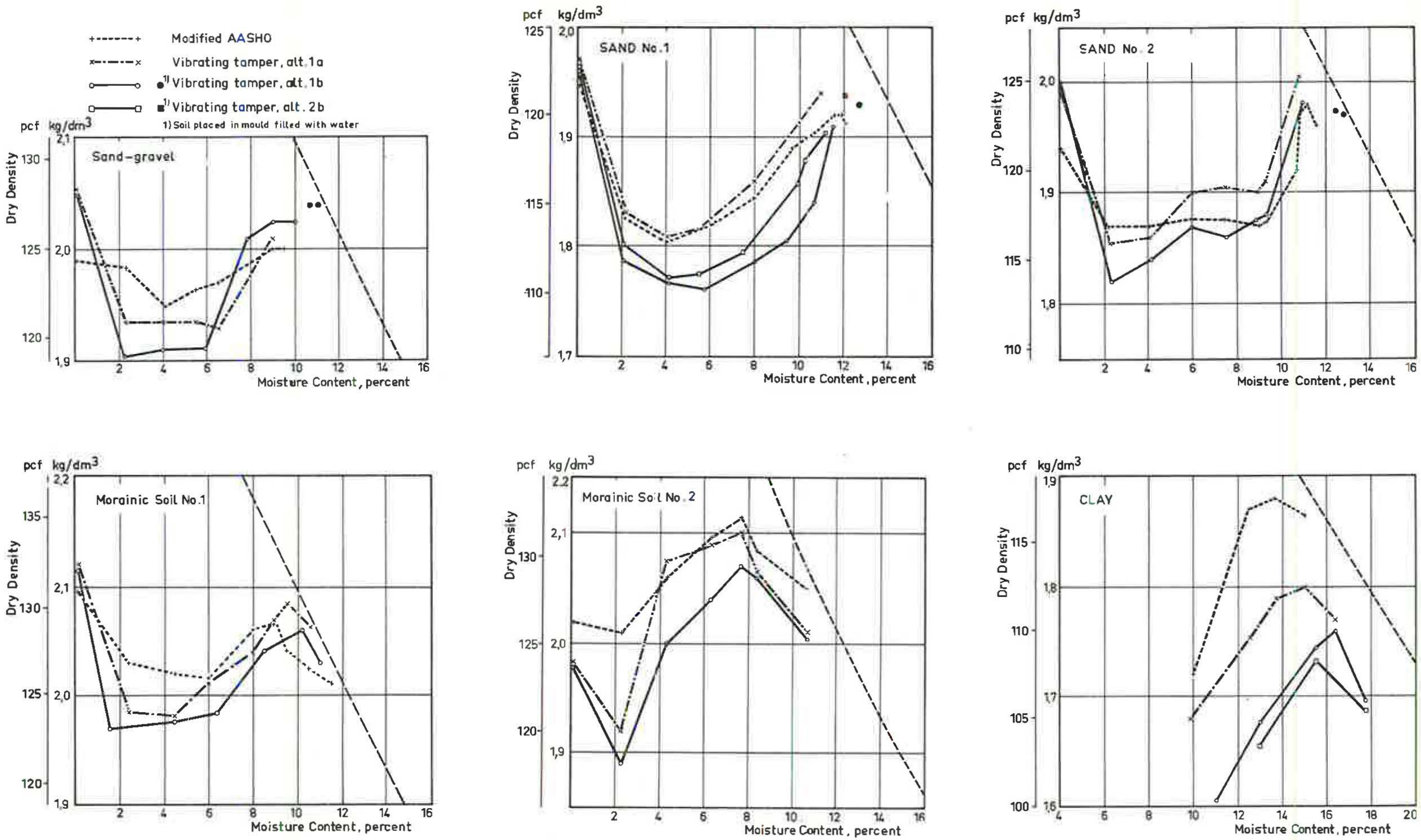

Figure 4. Results of laboratory compaction tests carried out by the modified AASHO method and the new method for laboratory compaction by vibration.

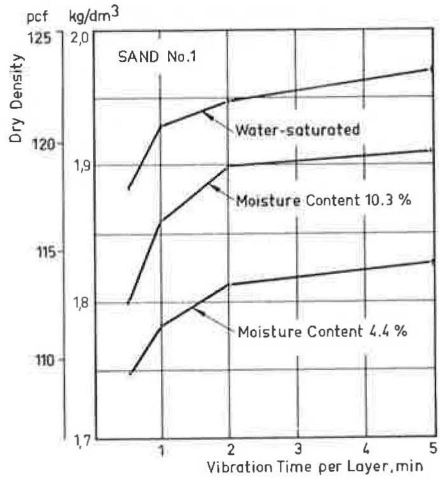

Figure 5. Relationships between dry density and vibration time obtained with apparatus defined in alternative 1b; vibration in two layers.

During the 1964 and 1965 working seasons this method was applied at the Håckren earth dam project in the northern part of Sweden. The part of the dam fill consisting of granular soils, 1.5 million cubic meters (2 million cu yd), varied from silty sand to gravelly stone. Density measurements were carried out in the compacted fill using a water balloon measure. A sample from each sampling point was vibrated in a saturated condition as described earlier under 1b. The density obtained in the laboratory tests was compared with the field density. The field densities averaged 96 percent of the lab values. Comparative tests by the modified AASHO method and the new laboratory method showed agreement to within ± 2 percent.

Despite the highly varied grading of the soil, the method allowed speedy and continuous checking of the degree of compaction of the fill. Granular soils containing varying quantities of stone are often used in bases and subbases for roads and airfields, and in fill used for building foundations. Thus there are several suitable applications for the new laboratory compacting method.

Study Stage 2

Experience during Stage 1 showed that the larger, 152-mm diameter (6 in.) mold is to be preferred, partly in view of the special usefulness of the method for stony cohesionless materials. Investigations of the effect of the time of vibration (Fig. 5) showed that a duration of 2 min instead of 1 min per layer gave closer agreement with the final results obtained in protracted vibration. The 2-min period is therefore preferable. Another aspect is that the 45-kg (100 lb) weight of the tamper made it difficult to handle. But if the duration of vibration is extended to 2 min per layer the weight of the tamper can be reduced; a further alternative was therefore studied.

(2b) Mold of 2.5 cu dm (0.09 cu ft) capacity

Diameter of mold 152 mm (6 in.)
Height of mold 138 mm ($5\frac{1}{2}$ in.)
Weight of tamper 35 kg (77 lb)
Frequency 3000 vib/min, centrifugal force 250 kg (550 lb)

Comparative tests as per 1b (1 min vibration per layer) and 2b (2 min vibration per layer) have shown good agreement in the results for granular materials tested at different water contents (Fig. 4). For the clay investigated, the light tamper gave a few percent lower maximum density than obtained with the heavy tamper. But this is no great disadvantage, and the light tamper, as in 2b, is to be preferred from the practical aspect.

CONCLUSIONS

A new laboratory compaction method based on vibration has been developed. The following equipment is recommended:

(2b) Mold of 2.5 cu dm (0.09 cu ft) capacity

Diameter of mold 152 mm (6 in.)
Height of mold 138 mm ($5\frac{1}{2}$ in.)
Weight of tamper 35 kg (77 lb)
Frequency 3000 vib/min, centrifugal force 250 kg (550 lb)

Vibration is in two layers, each layer vibrated for 2 min. In the United States and elsewhere with current of 60 cps, a vibrator with a frequency of 3600 vib/min is suitable. The corresponding centrifugal force must be tried out.

For granular materials, good agreement has been obtained between the new laboratory compaction method and compaction by the modified AASHO method. For cohesive materials, the new method gives somewhat lower figures. Sufficient basis exists for the recommendation of the method for practical use. The new procedure can also be used to compact test cylinders of soil-cement, asphaltic concrete, etc.

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Discussion

W. H. CAMPEN, Omaha Testing Laboratories, Inc.—This vibratory method of determining maximum laboratory density on noncohesive soils deserves consideration for two principal reasons: first, because methods are needed for the compaction of sands and gravels in the laboratory, and second, because this method may prove to be easier and more economical than the one which employs a vibrating table.

Although I believe this method has merit, I do not agree with some of the reasons Forssblad gives in its behalf. For instance, he emphasizes the good density correlation between his method and the modified AASHO method. If this were really true, it would detract from the method, since it is well known that the modified AASHO method is not suitable for round noncohesive soils because the material displaces under the foot, and consequently low density values are obtained. It would be well to correlate tests obtained by this method with those obtained by ASTM method D 2049, which is known to give very high density.

Forssblad also states that the method is good because density tests obtained with it correlate well with tests on sandy soils after compaction with vibratory field equipment. The reasoning appears to be faulty since it implies that density developed with field equipment is the basis for specifying maximum laboratory density. Maximum laboratory density, or some degree thereof, is used to indicate the structural strength of a compacted soil and is not related to the ease or difficulty with which it is developed in the field.

LARS FORSSBLAD, Closure—Campen agrees that a method for laboratory soil compaction based on vibration deserves consideration. He objects, however, to the correlations which are made between the new vibratory method and the modified AASHO method, but the author has not found any better method to correlate with the new method. An important reason is the common use of the modified AASHO method. With cohesive soils the new vibratory method has given lower values than the modified AASHO method. It is also likely that the agreement with the modified AASHO method will not be good for all types of cohesionless soils, as Campen indicates.

Campen's discussion contains the following sentence: "Forssblad also states that the method is good because density tests obtained with it correlate well with tests on sandy soils after compaction with vibratory field equipment." The following statement is made early in the report: "In cases where the field compaction is performed by vibratory compactors a laboratory test by vibration should give best correlation with field results." This sentence only indicates that a laboratory compaction method based on vibration is likely to give better results than other laboratory compaction methods when vibratory compactors are used for field compaction.

Experimental Study of Pulse Velocities in Compacted Soils

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Over 200 tests were performed on three basic soil types while studying experimentally the relationship between pulse velocity, dry density, water content, and compactive effort. Laboratory test specimens were prepared by kneading and impact compaction methods in split Proctor and Harvard molds, and in situ field tests were performed on the shoulders of a recently compacted highway embankment. All soils tested showed a monotonically increasing pulse velocity with increases in dry density until the optimum water content associated with a particular compactive effort was attained; then a rapid drop in pulse velocity was observed for further increases in dry density. The curve of peak velocities and the curve of maximum dry densities were approximately parallel and lie within ±0.5 percent water content of each other. Several factors are discussed which seem to influence the measured velocities. These are (a) size of the laboratory specimen, (b) type of compaction, (c) subsequent desiccation of the specimen, (d) method of defining the first arrival time of the pulse on the oscilloscope, and (e) the spatially dependent anisotropic macrostructure caused by the edge effects of the mold during compaction.

•THE stress-strain and strength properties of a soil are important in the construction of compacted embankments. Assuming that these properties are unique functions of dry density and water content for a given soil, specifications for the construction of compacted embankments normally require that each layer of soil be compacted to some stated minimum density and within a given moisture content range. To verify that the specified minimum density has been achieved, numerous in-place density tests are usually performed at random times and locations during the placement of the fill. Since the direct measurement of in-place densities by undisturbed sampling, sand cone, oil displacement, and rubber balloon methods is very time-consuming, engineers have attempted to correlate soil density with other more easily and more quickly measured physical soil parameters or indices, such as nuclear adsorbtion, electrical resistivity, and penetrometer resistance. The relative usefulness of these auxiliary methods for measuring soil density depends on the ease with which the two variables can be correlated and the extent to which this correlation is affected by changes in other variables, such as moisture content or electrolyte concentration. In this paper, a pulse velocity technique for measuring in-place soil dry density is studied and discussed.

The pulse velocity technique is already well established as a valuable engineering tool for quality control of many materials. Some of the more successful applications have been reported in the allied fields of concrete, asphalt, wood, metals and polymers.

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TABLE 1

SOIL PROPERTIES

Soil Sample	Specific Gravity (G)	Liquid Limit (W_L)	Plastic Limit (W_P)	Percent Finer Than			
				1 mm	0.1 mm	0.01 mm	0.001 mm
SM-1	2.74	32.1	17.2	95	86	62	31
SM-2	2.77	33.0	17.4	95	87	65	31
SM-3	2.76	27.0	16.8	91	72	54	25
SM-4	2.74	31.6	18.3	94	80	54	24
SM-5	2.76	34.9	17.6	97	88	66	33
CS	2.66	—	—	99	27	22	18
MC	2.72	53.0	23.3	99	91	68	39

For example, Long, Kurtz and Sandenaw (1) observed good agreement between concrete moduli determined by pulse velocity and static flexural testing methods. Similarly, Goetz (2) reported that the pulse velocity technique provided a relatively regular and uniform relation between wave velocity and asphalt content for different mixtures. Manke and Gallaway (3) claimed reasonable success in applying the pulse velocity technique to certain soils and bituminous mixtures. Wyllie, Gregory and Gardner (4) examined a heterogeneous mass of aluminum and lucite discs and showed that experimental velocities differed from theoretical ones, thereby indicating that theoretical conditions were not satisfied. Jones (5) showed that dynamic moduli of elasticity and layer thicknesses can be determined from field vibrational measurements, and McCoy (6) presented a single resonant frequency technique for determining shear wave velocity in infinitely large masses. Whitehurst (7) stated that pulse velocity provided a good criterion for comparing materials. In addition, Jones and Whiffin (8) presented an excellent survey of dynamic techniques for measuring the properties of pavement and subgrade materials, and summarized many of the divergent opinions expressed by researchers regarding the problems involved.

Although not specifically directed toward development of testing procedures, much work has been reported on wave propagation in soils. Leslie (9) investigated the velocity-water content relationship for a silty clay and found that maximum velocity occurred at maximum density and optimum moisture content. In their work on Ottawa sands, Hardin and Richart (10) studied wave velocities in saturated, partially saturated (drained), and dry sands, and forwarded certain concepts which Manke and Gallaway (3) later found to be consistent with their work. Utilizing the concept of logarithmic decrement, a study of the propagation and dissipation of elastic wave energy in granular soils was made by Richart, Hall and Lysmer (11) and Hall and Richart (12). Heierli (13) reported a similar study taking into account the complex nonlinear and inelastic stress-strain relationship for soils.

The transmission of impulses through a body has been investigated extensively for various geometrical configurations and idealized material properties. In any extended body, a generated impulse separates into a compression (or longitudinal or dilatational) wave and a shear (or transverse or distortional) wave. Under certain other conditions, surface waves may be generated at the interface between two different media, such as a free surface or the boundary between a layered system. These waves are categorized as Rayleigh or Love waves. Compression waves generally travel with the greatest velocity and are the only type considered in this investigation.

SCOPE OF TEST PROGRAM

Three basic soil types, a clayey sand (CS), a Vicksburg buckshot clay (MC), and a sandy silt (SM), were tested in this experimental program. Samples of the sandy silt were obtained at five different locations along the embankment approaches to the intersection of I-57 and I-80 south of Chicago; these are referred to as samples SM-1 through SM-5. The Atterberg limits, specific gravities, and grain size distributions of all samples are given in Table 1.

TABLE 2

SCOPE OF TEST PROGRAM

Compaction Device	Effort	Energy Source	No. of Layers	Tamps per Layer	Soils Tested	No. of Water Content Levels	Range of Water Contents (w %)	Range of Dry Densities (pcf)
Impact		10-Lb hammer, drops of						
	Lowest	12 in.	3	13				
		18 in.	3	25	SM-2	4	12-19	109-125
		18 in.	5	25				
	Highest	18 in.	10	25				
Kneading		Tank air pressure						
	Lowest	40 psi	5	16	SM-1	3	11-16	110-125
		65 psi	5	16	SM-2	3	12-16	105-124
		65 psi	10	16	SM-5	6	11-18	101-123
	Highest	70 psi	10	24				
Harvard Type A		Spring compression						
	Lowest	10 lb	5	3	SM-3	4	11-20	87-122
		20 lb	5	3	SM-4	4	11-18	80-115
		40 lb	5	6	MC	9	12-28	69-104
	Highest	80 lb	5	9				
Harvard Type B		Spring compression						
	Lowest	10 lb	5	3				
		20 lb	5	3	CS	4	4-15	79-118
		40 lb	5	3				
	Highest	80 lb	5	3				
Havard Type C		Spring compression						
	Lowest	10 lb	5	9				
		20 lb	5	9				
		40 lb	5	9	CS	4	4-15	83-127
		80 lb	5	9				
	Highest	80 lb	10	9				

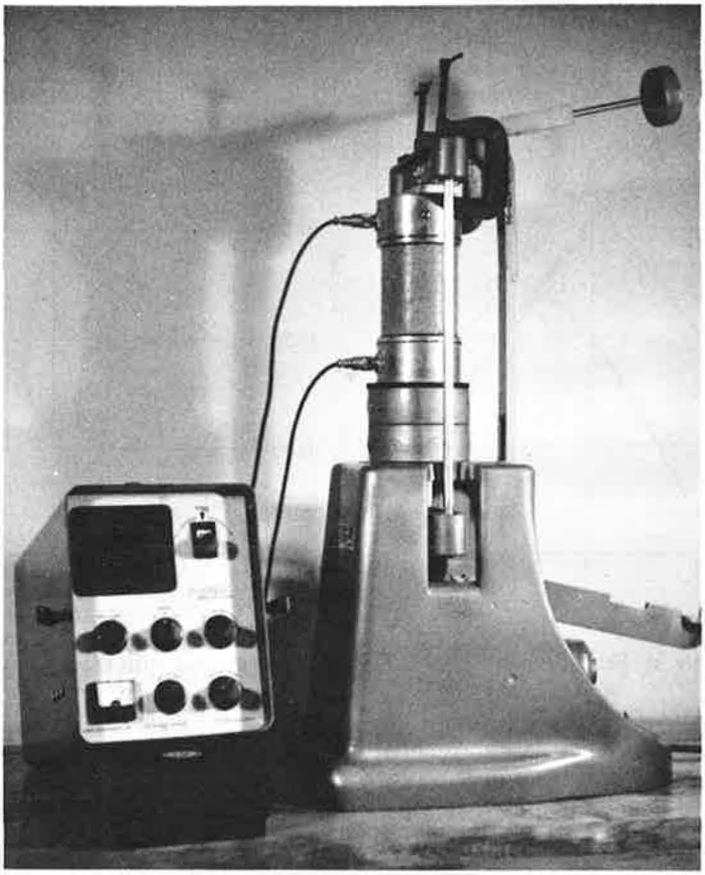

Figure 1. Test apparatus.

By using different types of compaction and several levels of compactive effort, over 200 test specimens covering a wide range of water contents and dry densities were prepared from these samples. The different types of compaction were accomplished by using a Havard miniature compaction device, a kneading or pneumatic compactor, and an impact or dynamic compactor. In the latter two instances, $\frac{1}{30}$ cu ft Proctor molds were used. The scope of the test program is summarized in Table 2.

EXPERIMENTAL PROCEDURE

Measurement of Pulse Velocity

The propagation velocity of a pulse through the compacted soil specimens was measured with a commercial electronic system which combines a pulse generator, source and receiver transducers, and an oscilloscope in one unit called a V-Scope. The source and receiver transducers consist of Rochelle salt crystals mounted in an aluminum case with a rubber membrane stretched over the open end; after the air is removed, oil is forced into the case under pressure. The pressure causes the rubber membrane to bulge beyond the end of the aluminum case so that it does not touch the specimen during the test.

The sweep rate of the oscilloscope is matched to the pulse frequency so that a stationary trace is displayed on the screen. The transducer tare time, which is the time required for the pulse to travel through a system with zero specimen length, is obtained by placing the source and receiver transducers in contact and reading the time

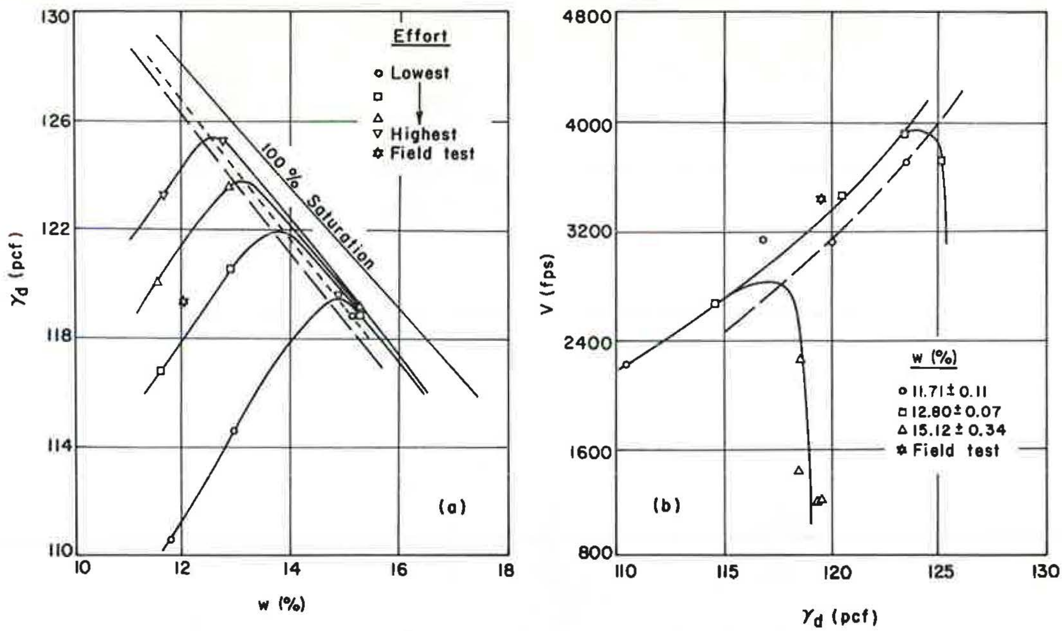

Figure 2. Results for SM-1 soil: Kneading compaction with Proctor mold.

on the oscilloscope. When calculating the time required for the pulse to traverse the specimen alone, the tare time is subtracted from the total time reading with the specimen in place. The pulse velocity is then obtained by dividing the gage length by the net pulse propagation time through the specimen. Figure 1 shows the V-Scope and a typical test setup.

Field Technique

The field testing was done on a compacted highway embankment which had been in place for about one month. Since the concrete lanes had already been placed, the shoulder areas were used; however, these areas had served as haul roads for heavy equipment and no check on the as-compacted densities was possible. Because Illinois requires no water content control, it was impossible to tell whether the embankment was compacted wet or dry of optimum or if significant wetting or drying of the soil had occurred subsequent to placement.

Using an 8-in. diameter earth auger, two holes were excavated about 12 in. apart and 16 in. deep. After the sides of the holes were trimmed to facilitate a greater contact area between the soil and the transducers, and to assure that the aluminum case did not touch the soil, the transducers were positioned about 12 in. deep and held tightly (exact force was not measured) against the soil by two small screw jacks placed against the back of the hole. Then, the distance between transducers and the oscillograph reading were recorded.

Following these measurements, the soil between the holes was excavated with a pick and a balloon density test was performed. In addition to water content samples, a minimum 20-lb sample of the soil was taken from between the holes for laboratory testing.

Laboratory Procedure

Laboratory specimens were compacted in either a split Harvard miniature compaction mold or a split $\frac{1}{30}$ cu ft Proctor mold; the interior of all molds was sprayed with a fluorocarbon dry lubricant. After the mold was removed, the transducers were positioned at each end of the specimen and the combination was subjected to approximately

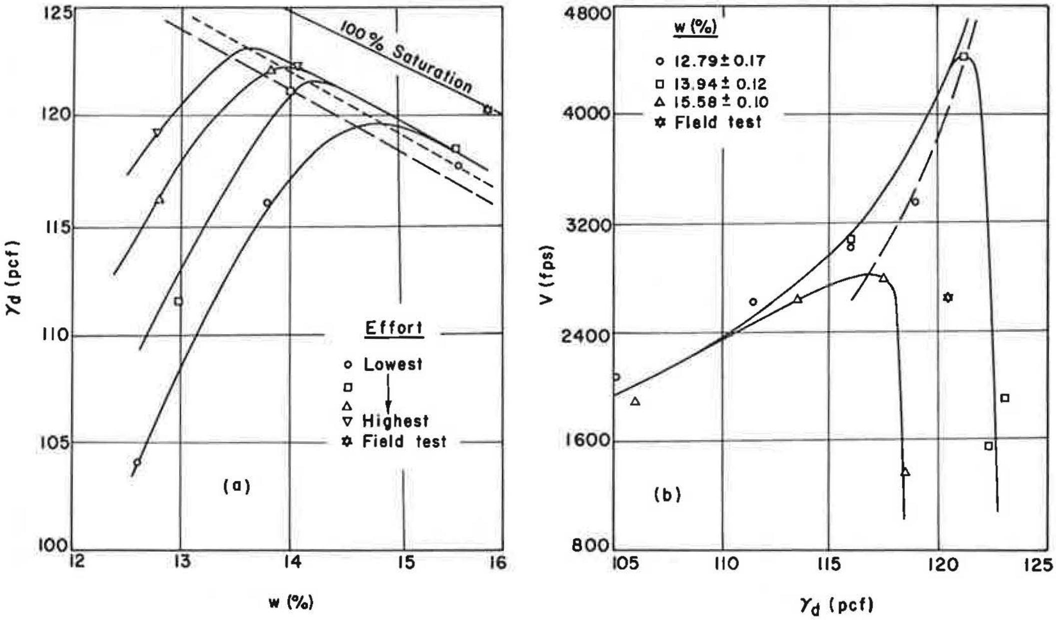

Figure 3. Results for SM-2 soil: Kneading compaction with Proctor mold.

3 psi axial pressure; this same pressure was used in determining the transducer tare time. A rubber pad was placed under the lower transducer to prevent waves from traveling through the apparatus and influencing the measurement.

For the specimens compacted in the Harvard mold, one velocity reading was taken in the axial direction; then, the water content was obtained by drying the entire specimen. For the specimens compacted in the Proctor mold, axial velocities were measured at five locations, once along the centerline and once at each of the quarter points near the edge of the specimen; the readings were then averaged. Water contents were obtained by splitting the specimens axially and securing a sample from the interior.

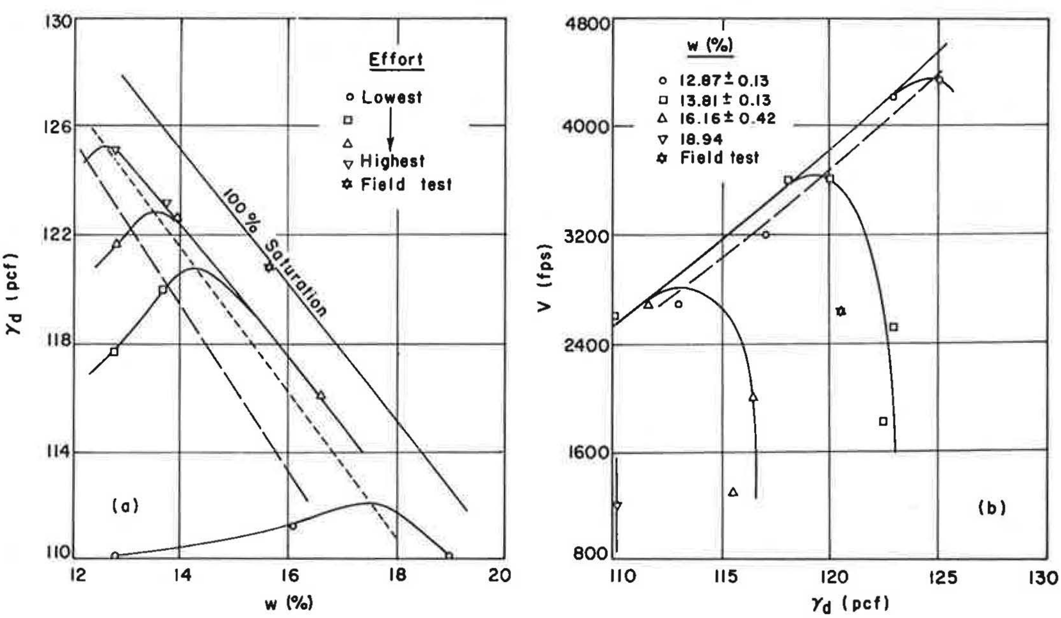

Figure 4. Results for SM-2 soil: Impact compaction with Proctor mold.

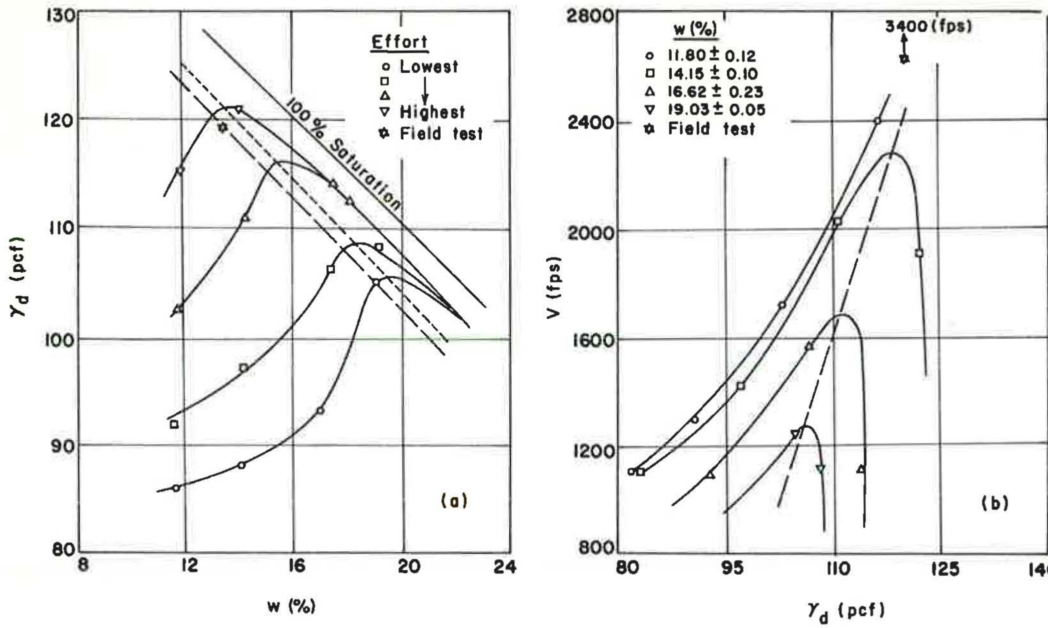

Figure 5. Results for SM-3 soil: Kneading compaction with Harvard mold.

EXPERIMENTAL RESULTS

The experimental results of all laboratory and field tests are given in Figures 2 through 9. Part a of each figure shows a plot of dry density, γ_d, vs water content, w, for several different compactive efforts. Part b shows the propagation velocity, V, plotted vs dry density for a relatively constant water content. In Part a of each figure a dotted line is drawn to connect the points of maximum dry density and optimum moisture content associated with each compactive effort. Similarly, in Part b the dashed line connects the points of maximum velocity. Finally, the dashed line connecting the

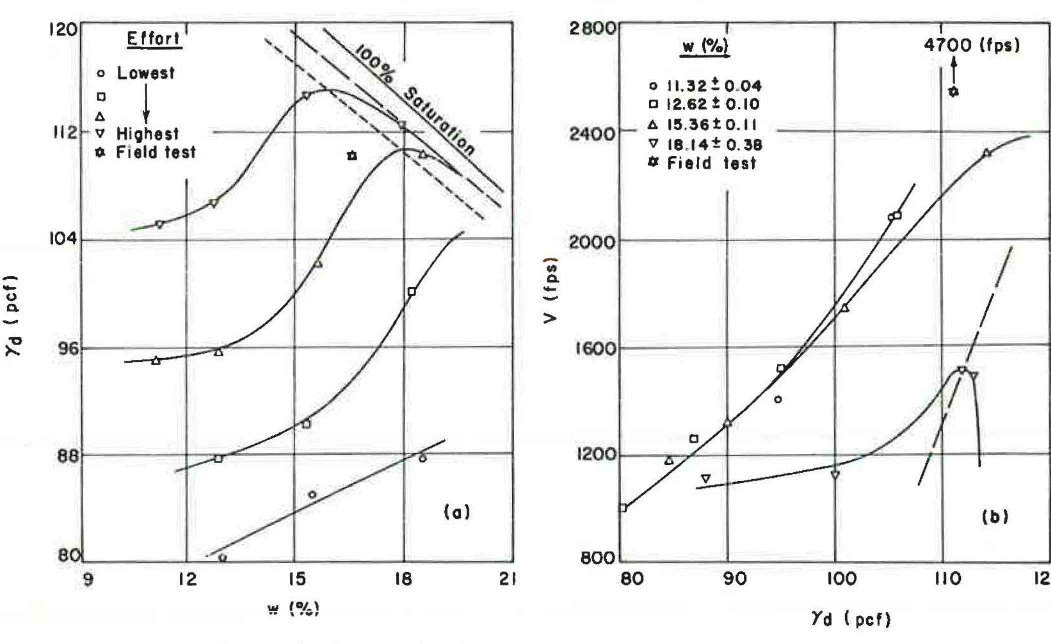

Figure 6. Results for SM-4 soil: Kneading compaction with Harvard mold.

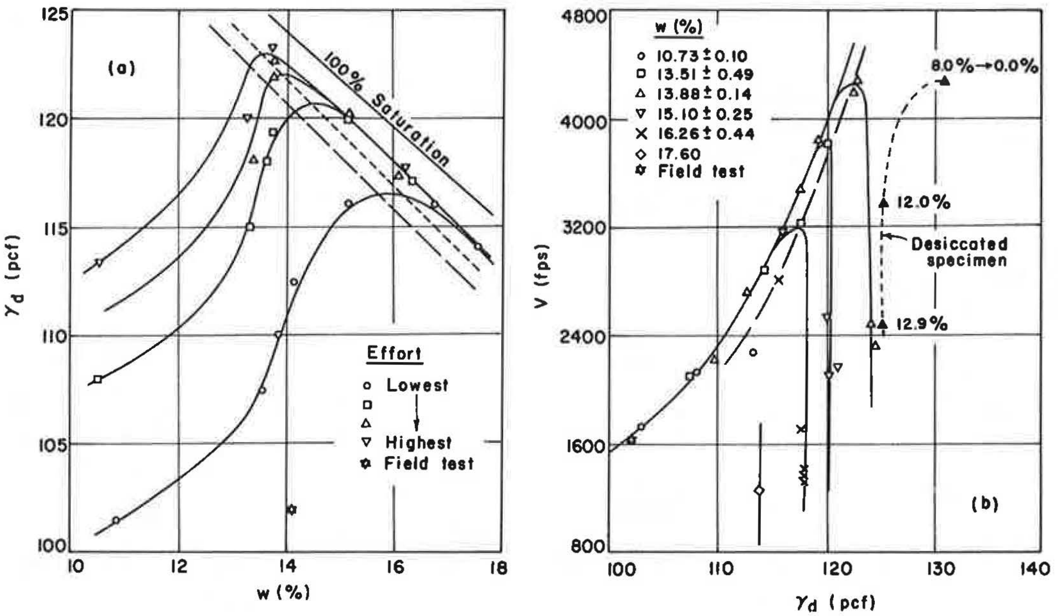

Figure 7. Results for SM-5 soil: Impact compaction with Proctor mold.

velocity peaks is superposed on the dry density-water content plot in order to evaluate its relationship, if any, to the associated dotted line connecting dry density peaks.

The following observations and interpretations are advanced in an attempt to explain and correlate the response obtained. When comparing the laboratory results of the five field samples, it was observed that the specimens compacted in the Harvard mold by kneading methods tended to yield velocities much lower than those of specimens compacted in the Proctor mold by both impact and kneading methods. Even though the densities obtained in the former case were generally lower, this does not completely

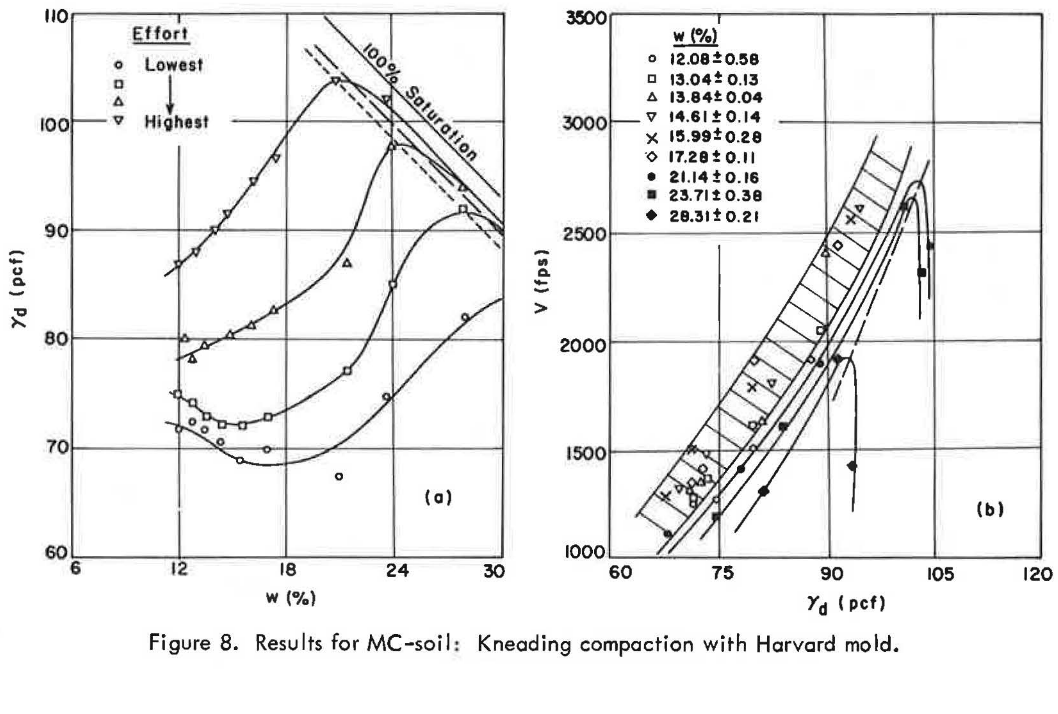

Figure 8. Results for MC-soil: Kneading compaction with Harvard mold.

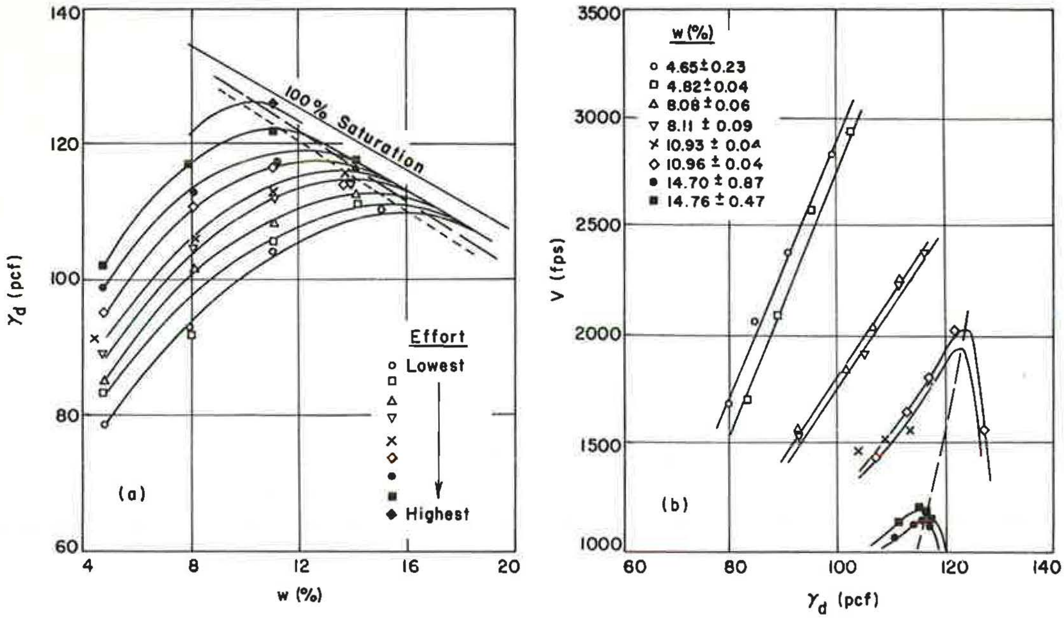

Figure 9. Results for CS soil: Kneading compaction with Harvard mold.

explain the discrepancy. Hence, in addition to possible effects due to variations in soil structure caused by different compaction methods, it appears that specimen size affects the propagation velocity; perhaps this is attributable to such causes as lateral inertial effects. Also, correlation with field data was poor for these specimens, as may be seen in Figures 5 and 6, while such correlation was generally good for the larger laboratory specimens compacted by kneading and impact methods.

As can be seen in Part a of Figures 2 through 9, the peak velocity curve approximately parallels the peak dry density curve and lies generally within 0. 5 percent water

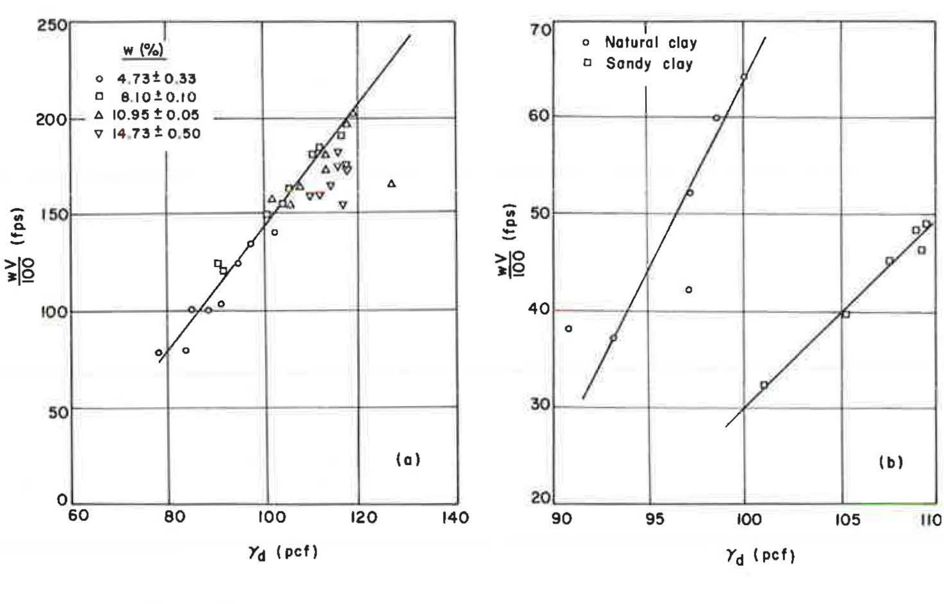

Figure 10. Product of water content and velocity vs dry density.

content either wet or dry of optimum. Three of the four samples compacted with the Harvard device had peak velocity curves falling wet of optimum, while the samples compacted in the Proctor mold by kneading or impact methods always had peak velocity curves on the dry side of optimum. A similar relationship between velocity and dry density peaks for a single compactive effort has been reported by Leslie (9) and by Manke and Gallaway (3). It seems apparent that the peak velocity is related to maximum dry density and optimum water content, and that some major soil characteristic which affects the compression wave velocity is changing at or near optimum.

There is some evidence that a change in structure occurs around optimum, but this structural alteration does not seem to be wholly responsible for the drastic velocity changes which were observed. Seed, Mitchell and Chan (14) have indicated that the structure developed in a compacted soil is greatly influenced by the shear strains induced during the compaction process. These strains apparently tend to produce a dispersed arrangement of soil particles in the region adjacent to the shear planes. Therefore, for soils in which the inter-particle forces are not so great that a random structure occurs under all conditions, compaction methods which induce large shear strains tend to produce a greater degree of particle orientation. Since relatively little shear deformation takes place in normal laboratory specimens compacted dry of optimum, soil fabric is not very sensitive to the method of compaction. However, for specimens compacted wet of optimum, particle orientation is dependent on the compaction method and tends to decrease in the order of kneading, impact, vibratory, and static compaction.

In the case of the clayey sand material, it was observed that the velocity-dry density data given in Figure 9 could be reasonably represented by a straight line for each water content dry of optimum. Furthermore, if these data are plotted in the form of the product of water content and propagation velocity vs dry density, all data dry of optimum tend to collapse empirically within reasonable experimental error into the straight-line relationship shown in Figure 10a. In addition, Figure 10b shows data taken directly from experimental results presented by Manke and Gallaway (3) and plotted in the same form. Moore (15) has presented the results of over 500 in situ field tests and over 50 correlation tests (mostly sand cone with a few oil displacement) from 8 or 10 different projects; approximately two-thirds of these were on granular base and subbase materials. Although no water content measurements were recorded, the velocity and dry density measurements taken under in-place field conditions seemed to indicate a straight-line relationship independent of water content. However, despite this fortuitous occurrence in the cases cited, it must be pointed out that such a collapse and straight-line relationship appears to be applicable only to the data dry of optimum. Furthermore, such a convenient and concise representation was not obtained for the other data in this experimental program, nor has it been substantiated by the works of many other researchers.

The results of an attempt to show velocity changes which occur as a compacted sample is desiccated are shown in Figure 7b. The specimen was compacted at 13.8 percent water content with the kneading compactor using the highest compactive effort; it was then allowed to dry to 12.9 percent, placed in a plastic bag, and stored in a humid room for several days to allow for a more homogeneous distribution of moisture. The velocity was measured at the end of this storage period. The same process was repeated at 12.0 percent, 8.0 percent and ovendry. At each point the dimensions of the specimen were measured, and these dimensions were used in the density calculations. The specimen appeared to be near the shrinkage limit at a water content of 8 percent. From these limited data it seems that water content below the shrinkage limit has little or no effect on the velocity. It is also interesting to note that the final velocity of the desiccated specimen is nearly the same as the maximum velocity indicated on the 13.8 percent curve; however, insufficient evidence is available to determine whether or not this is coincidental. The respective structures of the conventionally compacted specimen at peak velocity and of the desiccated specimen should differ significantly, since they were compacted with different compactive efforts and with moisture contents on different sides of optimum.

FACTORS INFLUENCING RESULTS

As with any experimental study involving relatively new techniques, there are a multitude of technical factors which influence measured results. While none of these factors were studied in sufficient detail to justify any conclusive statements, some qualitative comments are possible.

A definite problem exists in defining on the oscilloscope trace a single point which can be identified as the first arrival time of the pulse. In addition to the anticipated difficulty associated with defining the point where the slope of the trace is no longer horizontal, the reading varies with the input gain setting, since a higher gain setting magnifies that small amount of high-velocity energy which arrives ahead of the major portion of average-velocity energy. This same reasoning applies to the output energy setting. For example, very wet specimens displayed such a gradual slope change that the velocity could not be determined within 10 percent. The problem of defining the first arrival time in the field was compounded by the increased attenuation of the wave over a longer sample gage length. The amount of energy received was difficult to distinguish from the background noise in some cases. For very wet or very dry specimens, some problems were encountered in differentiating between a wave traveling through the air and one traveling through the specimen. Although a variation in axial pressure was found to affect velocity readings, this effect was not taken into account in this study.

When specimens were compacted wet of optimum, a definite spatially dependent anisotropic macrostructure was observed as a result of the soil being squeezed horizontally under the tamping foot. When the specimens were broken open, they exhibited a more-or-less radially-symmetric, saucer-shaped pattern. As a result of this orientation, the average propagation velocity near the edge was about 3 percent greater than that through the center; however, as previously mentioned, lateral inertial effects may also contribute to this phenomenon.

In order to study the directional dependence of velocity, two extra specimens were prepared; one was compacted wet of optimum, and the other was compacted as close to optimum as possible. After the axial velocity was obtained in the normal manner, the samples were trimmed into a square cross section with a slow-cutting power saw, thereby removing that portion of the specimen with the inclined planes. The axial velocity was again measured and found to correspond with previously measured values within about 10 or 20 ft/sec, well within experimental error. Then, velocity measurements were made in the perpendicular direction at several points. For the specimen compacted near optimum, the lateral velocity was about 2.5 percent greater than the axial, but for the specimen compacted wet of optimum, this difference was about 25 percent.

SUMMARY

The study consisted of pulse velocity measurements on over 200 specimens of three soil types compacted in the laboratory by kneading and impact methods in split Proctor and Harvard molds. The measured pulse velocities were related graphically with water content, dry density, and compactive effort. In addition, data from several field tests on one of the soil types were reported.

Based on these results, pulse velocity was found to increase monotonically with dry density for a constant water content until a maximum velocity was reached; thereafter, an increase in dry density resulted in a rapid decrease in pulse velocity. The curve connecting the peak velocities was found to approximately parallel and lie within ± 0.5 percent of the curve of maximum dry densities for several compactive efforts. For comparable water contents and dry densities, there were considerable variations between the velocities measured in the specimens compacted in the Harvard mold and those compacted in the Proctor mold. In addition to possible differences due to the method of compaction, lateral inertial effects may be more predominant in the smaller Harvard mold specimens, thus introducing a specimen size effect on the measured velocity.

Other factors influencing the pulse velocity measurements included specimen edge effects imposed by the mold during the compaction process and the anisotropy of the soil structure. For specimens compacted wet of optimum, the average velocity near the edge of the specimen was about 3 percent higher than the velocity at the centerline. Some preliminary tests on the effect of anistropy indicated that velocities in the lateral direction were approximately 2.5 percent higher than those in the axial direction for specimens compacted at approximately optimum conditions, and about 25 percent higher for specimens compacted wet of optimum. Indications are that desiccation drastically affects the measured pulse velocity and that the velocity in desiccated specimens is approximately constant for water contents below the shrinkage limit.

The pulse velocity technique has yet to be fully evaluated as an auxiliary method for measuring the in-place mechanical properties of compacted soil masses. Much work remains to be done in understanding the relationships between pulse velocities and other soil parameters and in developing techniques for obtaining pulse velocity measurements. However, if successful, this technique may offer some advantage over currently used controlled-density techniques. Because it can be directed toward the measurement of soil parameters which may be of more interest and importance than dry density in the construction of compacted embankments, the pulse velocity technique perhaps offers a method to circumvent the assumption that the mechanical properties of compacted soils are a unique function of dry density.

ACKNOWLEDGMENTS

The authors would like to thank Richard Muenow, Director of Ultrasound Research for James Electronics, Inc., Chicago, for providing the V-Scope used in the testing program. The cooperation of the Illinois State Highway Department in allowing field tests at the intersection of I-80 and I-57 is also appreciated.

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Correlation and Conference of Portable Nuclear Density and Moisture Systems

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This report summarizes the "Correlation and Conference" held in Charlottesville, Virginia, during the week of July 12, 1965. The Correlation and Conference had two phases: (a) the determination of the calibration curves for each device on prepared standards, and (b) field testing on a test road especially prepared for that purpose. Its purposes were to: (a) compare results obtained by the various portable nuclear systems used to measure densities and moisture contents of soils and aggregates; (b) attempt to reconcile any differences that might be found among the various systems; and (c) give those interested in conducting research with nuclear density and moisture apparatus the opportunity of conducting auxiliary experiments under controlled field conditions. The study was generally limited to portable surface gages that could test in either backscatter, direct transmission, or both; however, on request from a company that manufactures a vehicular mounted nuclear system, this equipment was also included.

•THE USE of portable nuclear gages for measuring the density and moisture of soils has advanced to the point that they are being utilized for compaction control by some state highway departments and are being observed with interest by others. Since about 1959 these density-moisture systems have been available commercially, and they are now being marketed by at least five manufacturers. In the use of these systems calibration curves are necessary for both the density and moisture gages. Because the various manufacturers obtain their curves on different standards, there is some concern over the relationship of one manufacturer's curve to those of the others. This problem was recognized in the course on Radioisotope Applications to Highway Engineering given by the Oak Ridge Insitute of Nuclear Studies in March 1964, where four manufacturers ran comparative density tests resulting in a variation of about 20 pcf(1). It was therefore felt that a correlation to reveal the variation that can exist between different devices and operators would be of great benefit to the users.

PLANNING THE CORRELATION AND CONFERENCE

Standards (Calibration Blocks)

In order to compare various nuclear systems, they must all be calibrated on the same standards or calibration blocks. Otherwise a basic level of comparison cannot be provided. The preparation of the moisture and density standards, therefore, constituted the most important part of the planning. After consultation, it was decided that a minimum of four (preferably five) density and moisture standards would be required to provide sufficient data for the development of calibration curves through statistical regression analyses.

Originally it was planned to obtain standards of the same chemical composition, but still covering a wide range of densities and moisture contents. It was thought this would

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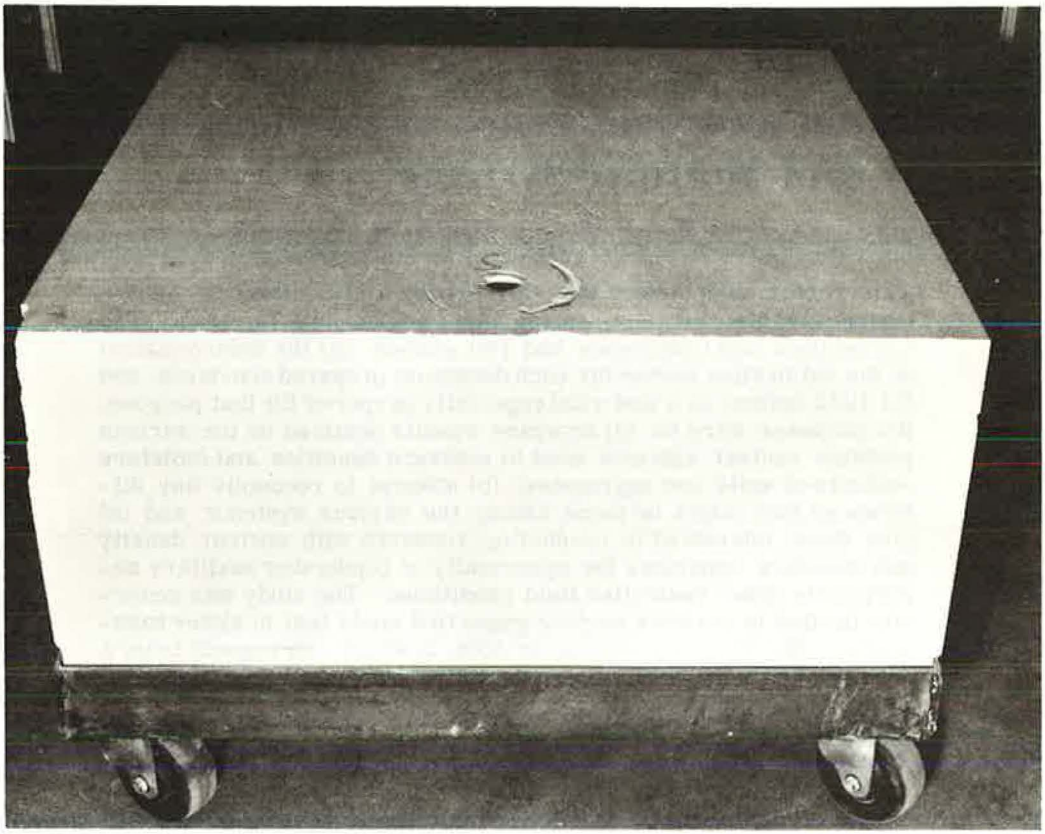


Figure 1. A typical density specimen.

be possible if glass standards of different densities could be obtained. The inability of glass manufacturers to produce such types of glass uniform enough for the purpose led to the abandonment of the idea. As will be explained later, this decision benefited the study since the effect of the chemical composition of the standards was a major factor contributing to the variablity of the results obtained.

TABLE 1

PROPERTIES OF DENSITY STANDARD BLOCKS

Property	Standard Number				
	I	II	III	IV	V
Description	Sand	Tufa	Chalk	Limestone	Granite
Wet density (pcf)	109.8	110.2	117.2	145.5	162.9
Percent absorption	–	24.3	10.7	4.7	0.2
Chemical analysis percent					
SiO_2	100.0	74.2	0.0	0.0	74.4
MgO		1.4	0.7	0.1	1.1
Al_2O_3		12.5	0.0	Trace	12.9
Fe_2O_3		1.4	Trace	Trace	0.7
Na_2O		3.3	Trace	Trace	3.0
K_2O		5.3	Trace	0.0	5.6
CaO		0.0	54.0	55.8	0.0
TiO_2		1.0	0.0	0.0	1.0
Loss at 800 C		0.4(H_2O)	43.9(CO_2)	44.3(CO_2)	0.4(H_2O)
Total	100.0	99.5	98.6	100.2	98.7

TABLE 2

PROPERTIES OF MOISTURE STANDARD BLOCKS

Property	Standard Number			
	VI	VII	VIII	IX
Description	Sand and Aluminum	Gypsum	Epsom Salt	Sand
Moisture (pcf)	12.2	21.6	34.2	0.7
Chemical analysis (%)				
Al_2O_3	2.4	–	–	See No. I
NH_3	0.7	–	–	
SiO_2	81.0	–	–	
SiO_3	6.7	46.5	32.5	
CaO	–	32.5	–	
MgO	–	–	16.3	
H_2O	9.5	18.6	51.2	
CO_2		2.0		
Total	100.3	99.6	100.0	

On the assumption that natural materials would be more uniform than manufactured materials, density standards were prepared from blocks of natural materials obtained from different parts of the country. These blocks were cut to a size 2 ft long, 2 ft wide and 1 ft deep—so dimensioned to eliminate boundary effects. Holes were drilled in these blocks to permit direct transmission measurements (Fig. 1). The blocks were mounted on casters so that they could be moved conveniently. Chemical analyses were run on the material from each standard block. Table 1 gives the properties of the density standard blocks. The densities varied from 110 to 163 pcf. It was thought desirable to extend the lower range to about 80 pcf and some materials were obtained for this purpose. However, the homogeneity of these materials was questionable and they were eliminated.

In preparing the moisture standards, primary consideration was given to moisture evaporation during use. It was therefore decided to prepare standards from materials having chemically bound moisture (or hydrogen). These materials were compacted into 2 by 2 by 1-ft lucite boxes to the desired density, which was the maximum attainable in most cases. The properties of the moisture standards are given in Table 2. It was desired to have five moisture standards, varying from 0 to 50 pcf; however, a value above 35 pcf was unattainable. In Tables 1 and 2 it should be noted that standards I and IX were the same; that is, sand was used as a density standard and also as a low moisture standard.

The homogeneity of some of the density and the moisture standards was relatively determined by taking a series of counts with a nuclear device. These counts were taken by moving the device after each count and covering an entire surface. As can be seen from Table 3, not all blocks were tested, and some were tested only from the top.

Test Road

To compare the performance of different devices (based on the calibration curves obtained on the standards), a five-section test road was built. Each section was 100 ft long, 12 ft wide, and 1 ft deep and was constructed with a different soil. These soils were chosen for their different chemical constituents and varying maximum densities. The soil properties in each section are given in Table 4.

TABLE 3

HOMOGENEITY OF STANDARDS

Standard No.	Description	Type of Standard	Coverage	Standard Deviation (pcf)
I	Sand	Density	Top	1.42
II	Tufa	Density	Top	1.75
III	Chalk	Density	Overall	2.14
IV	Limestone	Density	Overall	0.96
VIII	Epsom Salt	Moisture	Top	0.87
IX	Sand	Moisture	Top	0.33

TABLE 4

PROPERTIES OF SOILS USED IN THE TEST ROAD

Property	Section				
	A	B	C	D	E
Description	Micaceous silt	Limestone residual clay	Greenstone residual clay	Clayey sand	Crushed stone (top size 2 in.)
Maximum density of -4 portion (pcf) 100	100	94	84	127	134
OMC of -4 portion (%) 23	23	26	35	8	8
Atterberg limits					
LL (%)	43	68	60	NP	20
PI (%)	NP	33	2	NP	NP
HRB classification	A-5(3)	A-7-5(20)	A-5(12)	A-1-b	A-1-a
Specific gravity	2.66	2.70	2.90	2.66	2.77
Gradation, percent passing sieve					
1 in.	—	—	—	100	83
No. 4	—	—	—	73	—
No. 10	100	100	100	53	35
No. 40	77	95	97	26	20
No. 200	48	89	85	10	11
Chemical analysis (%)					
SiO_2	54.5	66.9	40.0	83.3	60.0
MgO	0.1	0.7	1.0	1.2	2.3
Al_2O_3	16.8	14.9	20.1	5.0	16.0
Fe_2O_3	6.9	7.8	19.8	2.4	10.7
Na_2O	0.0	0.0	0.7	4.0	3.0
K_2O	0.1	Trace	0.6	2.5	5.6
CaO	0.0	0.0	3.0	0.0	1.7
TiO_2	0.0	0.0	2.3	0.0	0.0
Loss 300C (H_2O)	19.3	6.9	12.8	1.1	1.6
Total	97.7	97.2	99.2	99.5	100.9

In each section, eight frames were built to mark the sites for making measurements. Frame locations were randomized within a section. The surface of the soil within these frames was smoothed to eliminate most of the "seating" effects. Finally, the test road was covered with plastic sheeting to minimize changes in the moisture content.

Procedures

Twenty-six agencies from state highway departments, schools, research organizations and equipment manufacturing firms throughout the nation attended the Correlation and Conference either as participants or observers. Some foreign representatives were also present.

At a briefing session the participants were assigned code (or gage) numbers and supplied information on the testing program. They were then asked to calibrate their devices, in a randomized manner, on the standard blocks prepared for that purpose. For the purpose of statistical analyses, the participants were asked to make four repeat measurements on each block without moving their devices.

During calibration, the standards were spaced about 10 to 12 ft apart. This distance was thought sufficient to eliminate interference from one device to another. However, some participants claimed that they detected radiation from other gages, indicated by a great change in magnitude in the counting rate. Therefore, some standards were moved to other locations and more distance provided between them. As a result, some recalibration was necessary.

Through the use of a computer and the data obtained from calibration on standard blocks, calibration curves for density and moisture were developed for each device. After the calibrations were performed, the participants were asked to make field measurements on the specially prepared test road. Again for statistical analysis purposes each participant was asked to make two repeat measurements without moving his device on each of the eight sites in each of the five sections. To convert the counts or count ratios obtained in the field to actual densities and moisture contents, the calibration curves drawn from measurements made on the standard blocks were used. This was done with a computer. During the meeting the participants were asked to assemble after testing whenever possible. These conference sessions were used to discuss some of the problems that arose and also to collect or distribute data.

ANALYSIS AND RESULTS

From the standpoint of interest, it would seem in order to discuss the analysis and results of wet densities first and then to discuss the moisture measurements, because it is generally recognized that the variations found in the latter are less serious than those in the former.

The analysis excluded a few gages found to be malfunctioning. Gage 17 (a special experimental design) did not participate in field testing and thus was excluded from the entire analysis. Density gages 6 and 22 were found to be malfunctioning on some test sections and thus were not analyzed on those sections. Gages 19 and 29 measured only in direct transmission for density. Moisture gage 18 did not standardize properly and thus was excluded from the moisture analysis. Gages 19 and 29 did not have time to make moisture readings. In some other cases readings were found that looked questionable, but since no valid reason could be found to eliminate them, they were not excluded from the analysis.

It is apparent that a great deal of information can be gleaned from the voluminous data obtained in the study. However, much of this information is of vital interest only to the particular participant involved. Therefore, the following discussion, as much as possible, involves general trends and overall conclusions. It should be noted that whenever a trend is mentioned a possible reason is also mentioned. It is left up to each participant to evaluate his own gage, and it is hoped that this will be done in every case.

Wet Density Calibration

Wet density calibration curves were obtained for 30 gages using backscatter measurements, 11 for direct transmission (at a 6-in. depth) and two for air gap. A typical calibration curve is shown in Figure 2, with the five calibration block densities plotted

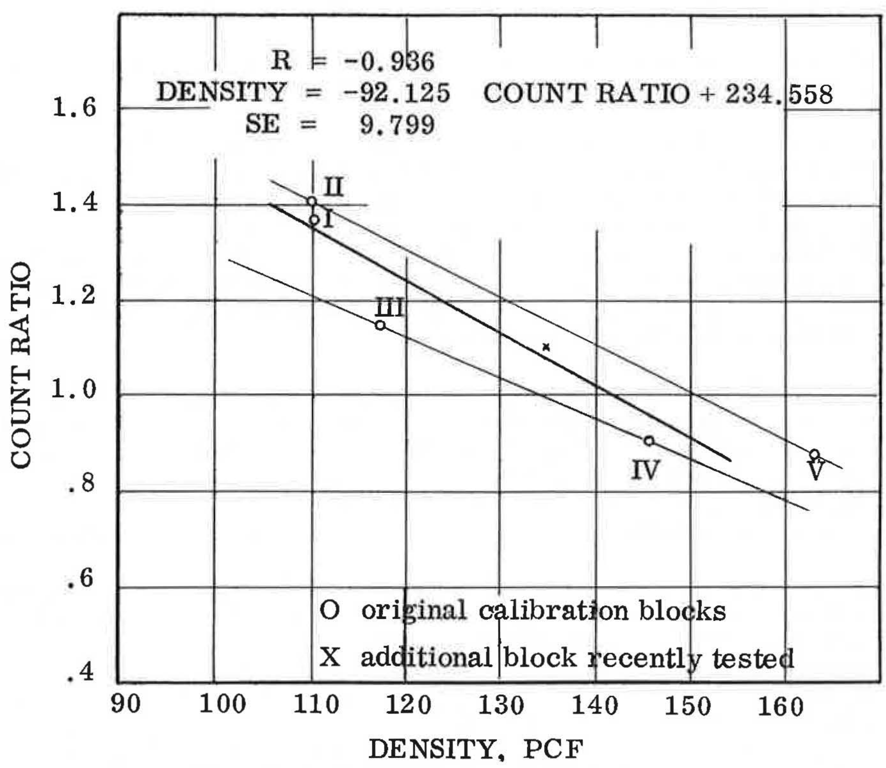


Figure 2. Typical wet density calibration curve.

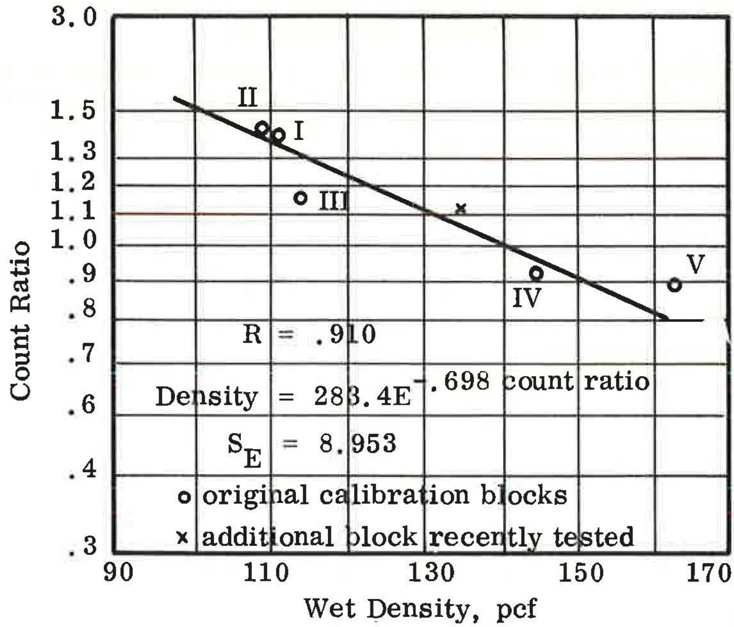


Figure 3. Semilog plot of typical wet density calibration curve in Figure 2.

against their corresponding count ratios. (Count ratios rather than counts were used in every analysis in this report.) Also shown are the correlation coefficient, the equation and the standard error (an expression of variation and not a mistake—the smaller the standard error, the closer are the data points to the curve) for the linear regression line.

The distribution of the points in Figure 2 raised a question concerning the shape of the curve to be used. Two possibilities were considered:

1. The curve should be a smooth sag curve that goes through the five original points. This would have a small standard error and high correlation coefficient, but would have the disadvantage of poor sensitivity above a density of about 145 pcf.
2. The curve should be linear and the departure of the points from a linear function is due to the difference in chemical composition between blocks III and IV and blocks I, II and V. According to nuclear theory, calcareous materials (blocks III and IV) should have a lower count ratio for a given density than siliceous materials (blocks I, II and IV), which is exactly what Figure 2 indicates.

Two additional steps were taken to help clarify the answer to this question. The first was to use regression analyses other than linear. This was done by the Florida Road Department. Figure 3 (which is Figure 2 plotted on semilog paper) indicates that a semilog plot would remove some of the variations existing in the linear regression as evidenced by smaller statistical error. However, this analysis also indicated that a log-log plot as well as a hyperbolic function produce curves that are closer to the data points.

The other step was to obtain a block of siliceous material with a density between that of the calcareous materials. A pyrex glass block, essentially 100 percent silica with a density of 135.4 pcf, was purchased for this purpose. This block, denoted by an x in Figures 2 and 3, is shown for descriptive purposes only and was not included in any analysis. It was tested by only one gage (15) and is therefore only directly applicable to the behavior of that one gage. However, it can be seen in Figure 2 that three lines can be drawn through the six data points, one for blocks II and IV (75% SiO_2 and 13% Al_2O_3), one for block I and new glassblock (100% SiO_2) and one for blocks III and IV (55% CaO). This would tend to reinforce the original conclusion that the curve should be linear and the departure of the points from linearity is due to the difference in chemical composition between the blocks.

Another question on this same general subject concerned the possibility that different gages might require different regression functions. This possibility seems reasonable from the standpoint of both different geometry and different sources from gage to gage. The data taken from the regression analysis done by Florida indicated that every gage but two (gage 19 in direct transmission and 6 in air gap) had standard errors ranked in the following order, with the average standard error shown:

1. hyperbolic $Y = X/B + AX$ SE = 6.3
2. log-log $Y = AX^B$ SE = 7.6
3. semilog $Y = Ae^{BX}$ SE = 9.0
4. linear $Y = A + BX$ SE = 9.8

Of course what has happened is that the functions with the greatest curvature have provided the best fits. But the interesting aspect of this analysis was that nearly all the gages were improved about equally by changing from a linear to semilog to log-log form of regression analysis.

The authors believe the relatively large magnitudes of the standard errors, particularly in backscatter, are due to a great extent to the chemical effect, which varies considerably from block to block (2, 3). R. P. Gardner of the Research Triangle Institute has indicated that by taking the chemical effect into consideration the standard error is decreased to about 2 pcf for the gages in Table 5.

However, it is recognized that other factors may have added to the variations. These are (a) density variations within the blocks, (b) background radiation variations present while calibrating, and (c) the use of linear regression rather than a semilog equation. It is believed that these three factors do not contribute to the total standard error as much as does the chemical effect.

TABLE 5

STANDARD ERRORS FOR WET DENSITY CALIBRATION CURVES

Gage No.	Backscatter (pcf)	Direct Transmission (pcf)	Air Gap (pcf)
1	10.1		
2	10.5		
3	10.5	8.4	
4	7.3		
5	12.8		
6	13.9		2.8
7	13.2		
8	10.1		
9	9.7	6.4	
10	10.0		
11	11.2		
12	9.5		
13	9.6	7.7	
14	8.6		
15	9.8	5.7	
16	9.3	5.8	
18	10.5	7.0	
19	13.6	4.4	
20	13.1		
21	10.8		
22	11.1		2.4
23	10.7		
24	13.5		
25	10.8		
27	15.6		
28	11.3	15.3	
29		7.1	
30	11.4		
31	7.4		
32	14.8		
35	9.8		
Average	11.0	7.53	2.6
Standard deviation	2.0	3.1	0.8

Some general trends can be interpreted from Table 5:

1. The standard errors with the backscatter technique are fairly constant from gage to gage. There are one or two extreme values but most of the values are close to the mean.

2. The direct transmission standard errors are generally less than the corresponding backscatter values. This could be caused by the fact that direct transmission is affected less by chemical effect than is backscatter; the 6-in. depth density is closer to the overall calibration block density; and/or background radiation was not as pronounced during calibration. (It should be noted that fewer gage designs are included in direct transmission than in backscatter.) Gages 6 and 22, with standard errors of 22.4 and 22.7 pcf respectively, could calibrate on only four of the five density blocks and thus did not lend themselves to the analysis.

3. Although there are only two results for the air gap technique, the standard errors are quite low as compared to the other values.

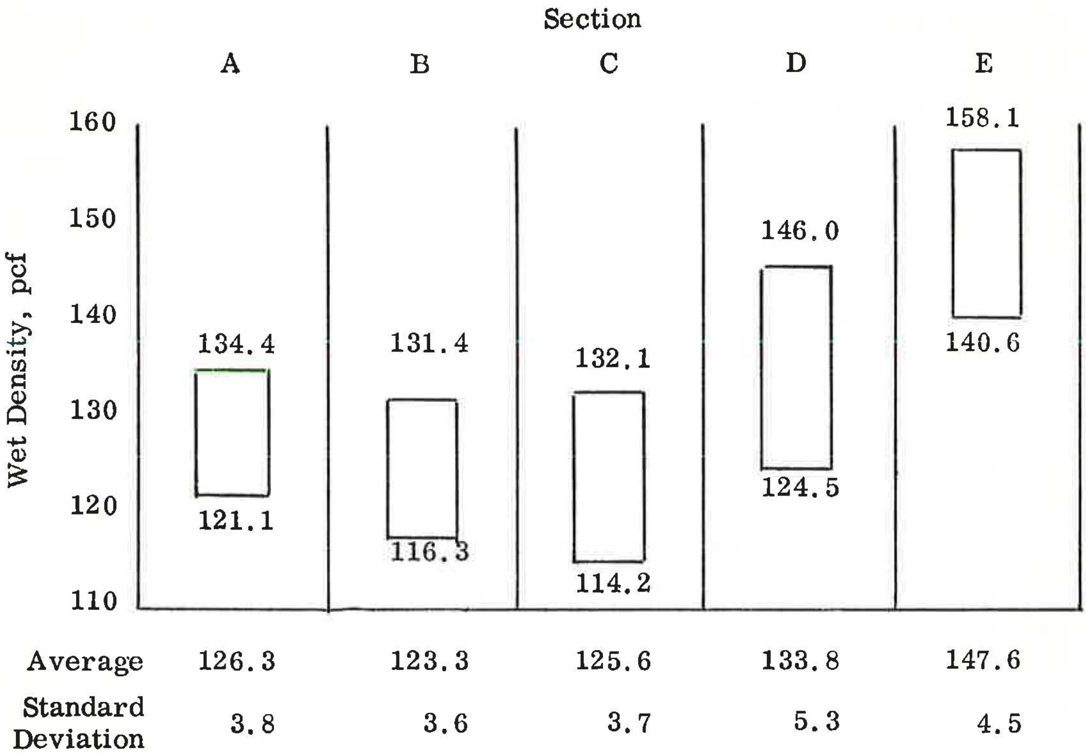


Figure 4. Variations found in average wet density values among all 29 gages using backscatter measurement.

Wet Density Measurements on Test Sections

Backscatter—The linear calibration curves produced were used to convert the count ratios found on the test sections to wet density. Figure 4 shows the variations found within each section. The average wet density of each gage, in rank order, for each section is shown in Appendix A.

These maximum variations of from 13 to 22 pcf among 29 gages should not be surprising when consideration is given to the variables included in the data. The gages investigated would be a good sample of the entire commercial production throughout the United States. These gages vary in age as much as six years. Some are more susceptible to background radiation and/or chemical effect than others. Some were used in techniques in which they are not normally used. And there is always the possibility of unnoticed malfunctionings.

There are two characteristics, both shown in Figure 4, that should be examined to obtain the most information from the data. One is the standard deviation. As Figure 4 shows, the values for sections A, B and C are less than for D and E. This would indicate that the chemical effect is not as severe from gage to gage as is the particle size effect. This reasoning is based on the fact that the iron is relatively high in A and B and very high in C, which should definitely provide an unusually high chemical effect; however, all gages measure it at a remarkable uniformity. Sections D and E have much coarser gradations than A, B and C, as shown in Table 4 and evidenced by the surface textures shown for each section in Figure 5; this factor may have caused "seating" variations.

An attempt was made to decrease the standard deviations in Figure 4 by using separate calibration curves based on matching chemical analysis of calibration blocks to test sections as closely as possible. This attempt in no way aided the analysis and thus

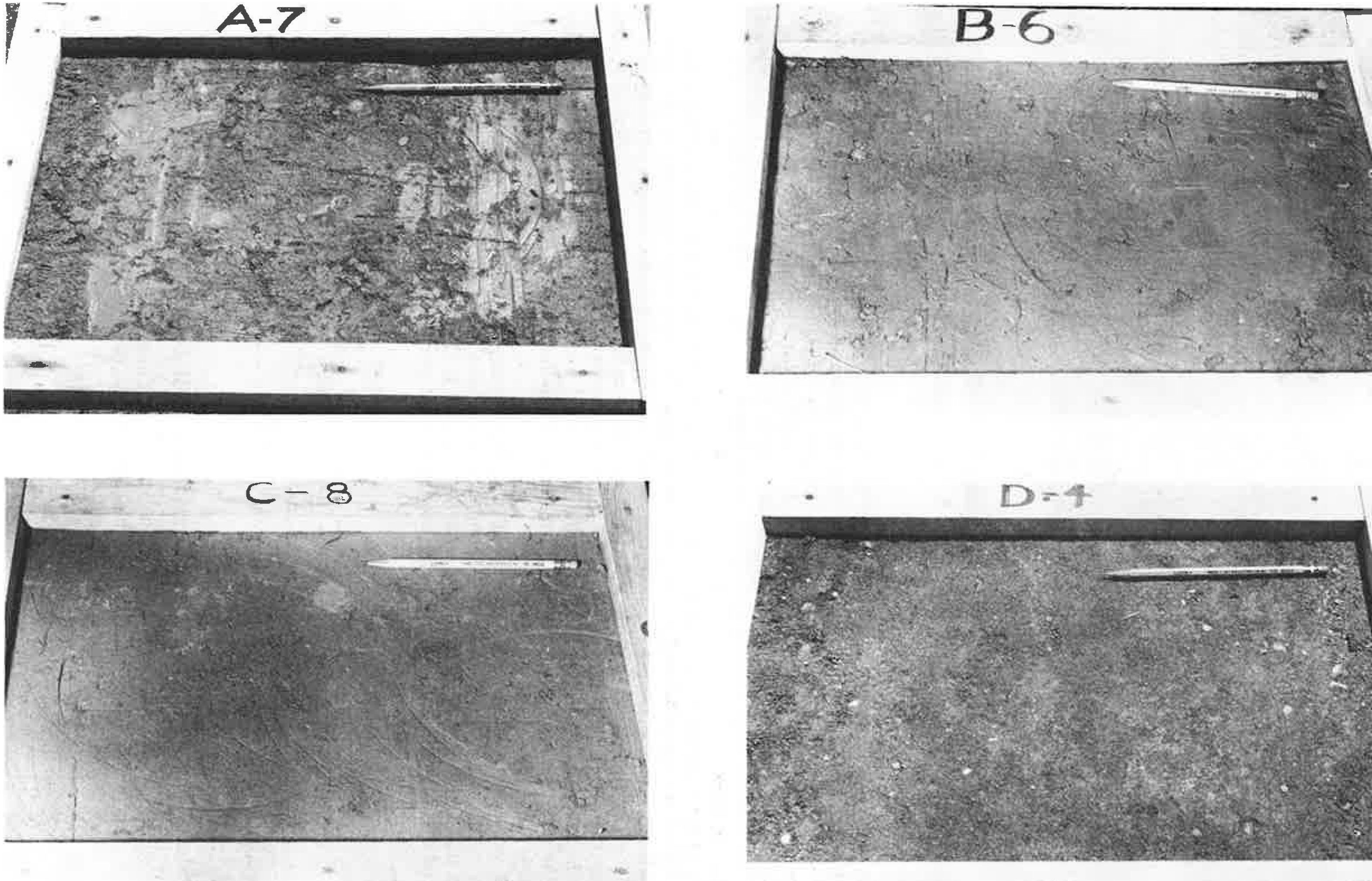


Figure 5. Typical surface texture photographs of test sections.

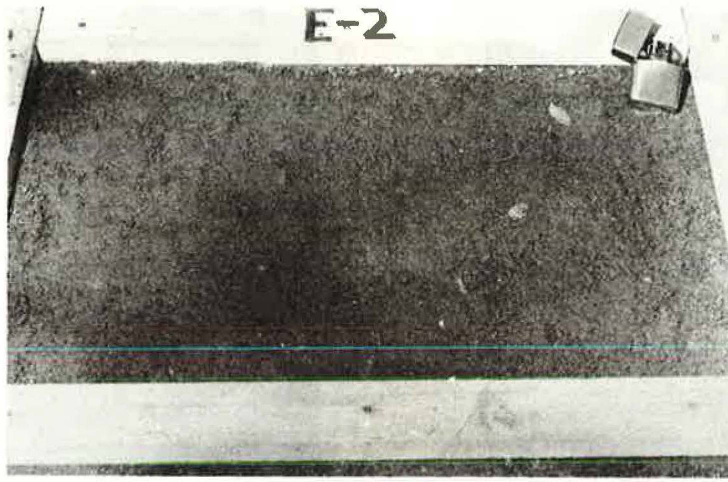


Figure 5. (Continued).

is not shown. However, an attempt was made to help answer the question concerning the effect of the age of the gages on the results shown in Figure 4. The results of gages known to be not more than two years old (gages 5, 9, 11, 12, 14, 15, 16, 22, 24, 30) decreased the ranges found in Appendix A to 4.8 pcf for section A, 4.1 for B, 7.9 for C, 5.9 for D, and 5.6 for E. This would tend to indicate that the changes in gage designs over the years affect the results of this report and that newer gages produce fairly close results.

The other characteristic which should be examined in Figure 4 is the overall average density value obtained on each section. When these values are compared to the wet densities that are obtained at maximum dry density and optimum moisture content (Table 4), it is evident that the wet densities measured are very near those that should be expected for all sections except C. This may be because the calibration curves did not take into account the high iron found in this test section. This would again indicate that the chemical effect is important; more so to nuclear measurements in general, however, than to any particular gage.

As previously mentioned, in Appendix A the average wet density is listed for each gage, and from this information a comparison between gages is possible. It was decided that the analysis should go deeper into trying to determine why the differences existed and also to show statistically how well each gage compares with every other

$$\sigma_t = \sqrt{\frac{16.0}{N} + \frac{0.7}{n}} \qquad \sigma_{ws} = \sqrt{\frac{0.7}{n}}$$

$$\sigma_{bs} = \sqrt{\frac{16.0}{N}}$$

(Values shown are actual average variations of all gages on all sections)

σ_{bs} = between-site variations (variability of density of material)
σ_{ws} = within-site variations (variability of nuclear gages)
σ_t = total variation as measured
N = number of locations tested
n = number of tests at each location

Figure 6. Sources of variation in density testing by nuclear gages.

TABLE 6

NUMBER OF SECTIONS ON WHICH NO STATISTICAL DIFFERENCE WAS FOUND BETWEEN GAGES
(Wet Density, pcf, Backscatter Technique)

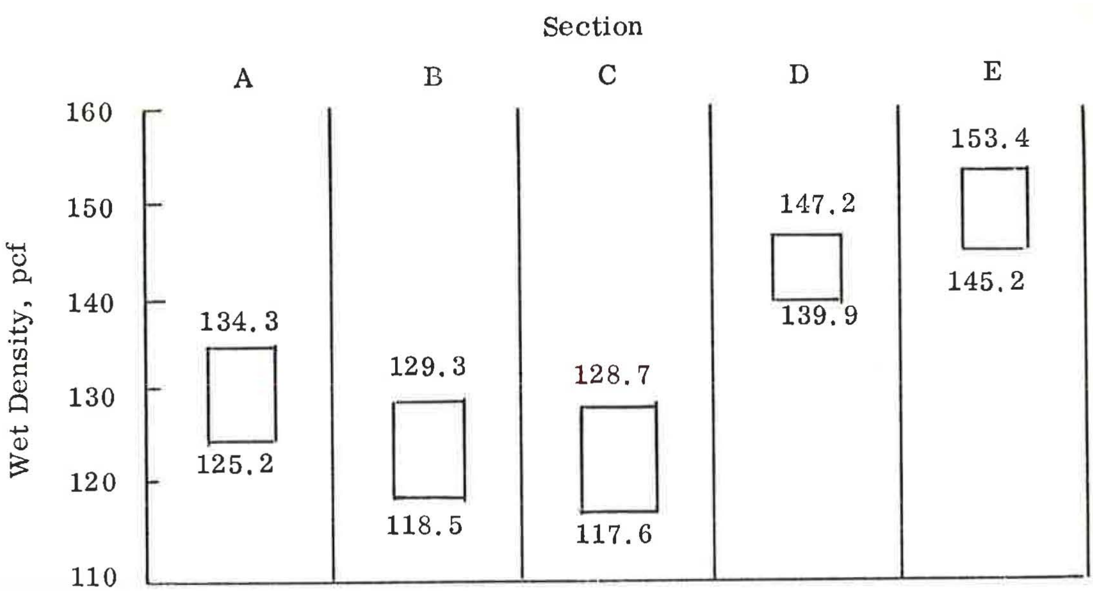
Gage No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	22	23	24	25	27	28	30	31	32	35
1		5	0	5	3	2	1	5	3	1	3	1	5	4	3	2	0	1	5	0	4	3	3	1	3	5	5	3	3
2			0	5	2	1	1	5	2	5	0	0	4	2	0	0	0	0	3	0	4	2	4	1	0	3	4	1	2
3				0	3	3	3	0	3	0	4	4	1	1	1	4	3	2	0	2	0	2	0	2	3	1	0	3	2
4					4	1	1	4	3	1	1	0	5	3	4	2	0	1	4	0	3	3	3	1	3	3	5	1	2
5						2	2	3	3	0	4	3	4	5	4	4	0	1	5	2	3	5	2	2	5	5	3	4	2
6							4	1	1	0	4	3	1	2	3	2	3	4	1	1	1	4	0	4	3	3	1	4	2
7								1	1	0	4	3	1	2	2	3	4	5	1	1	1	3	0	5	3	1	1	4	1
8									2	2	0	0	4	2	2	1	0	0	3	0	4	3	3	1	1	3	4	2	2
9										0	2	4	4	5	5	5	0	1	5	2	3	3	1	2	5	4	3	2	3
10											0	0	0	0	0	0	0	0	0	0	1	0	4	0	0	0	1	1	1
11												4	1	4	3	5	3	4	2	2	1	5	0	3	4	3	1	4	1
12													0	3	4	5	2	3	1	2	1	4	0	2	4	1	0	4	2
13														5	5	4	0	1	5	1	4	3	2	1	4	5	4	3	2
14															5	5	0	1	5	2	4	4	1	2	5	5	3	2	2
15																5	0	1	5	2	4	5	1	2	5	5	3	2	3
16																	1	3	5	2	4	4	1	2	5	5	1	2	3
18																		4	0	1	0	1	0	4	2	0	0	3	0
20																			1	0	1	1	0	4	2	2	1	4	1
21																				1	4	3	1	1	5	5	4	2	3
22																					0	2	0	0	2	2	0	2	1
23																						2	2	2	4	4	4	2	2
24																							0	2	4	4	3	5	2
25																								0	1	1	2	1	1
27																									2	1	1	2	0
28																										4	1	2	2
30																											3	3	3
31																												0	2
32																													2

gage. In Appendix B is an anlysis of "between-site" variations and "within-site" variations (between repeat tests) for each gage. It can be seen that in every case the between-site variation is greater than the within-site variation, and usually quite a bit greater. In fact, only four out of a possible 145 section results (5 sections times 29 gages) indicated no statistically significant difference from between-site and within-site variances. What this means is that the variations found in these four instances were of such magnitude that the between-site variance may have been larger than the within-site variance by chance rather than any real cause. Conversely, however, on the remaining 141, the between-site variances were significantly larger than the within-site variances.

Without going into nuclear theory, but by only taking the variations obtained from the 29 gages, the practical implications of this finding are very important. Figure 6 shows the average variations found between sites (σ_{bs}), within sites (σ_{ws}), and the total (σ_t).

It can readily be seen that any attempt to reduce σ_{ws} will have little effect on σ_t. On the other hand, if σ_{bs} is reduced by a factor of two, almost all of the reduced variation is reflected in σ_t. Therefore, from a time standpoint it is much better to increase N (test more locations) and let n = 1. This produces a much better estimate of the total variation present than does the taking of repeat tests. Based on the analysis shown in Figure 6 and the preceding discussion, it should be agreed that one reading per site is sufficient and thus allows much more testing for a given period of time.

To make it somewhat simpler to compare gages on the test sections, Table 6 was prepared, which gives the number of sections on which each gage compared with every other gage. The number in the block indicates the number of sections on which no statistical difference was found and was derived from Appendix C. This was done by a statistical technique called wholly significant difference (WSD) for testing the multiple differences between a number of means (4). Briefly, this method considers the number of gages being compared, the number of replicates for each gage and the pooled estimate



	A	B	C	D	E
Average	129.2	123.1	122.7	142.9	151.1
Standard Deviation	2.7	2.9	3.5	2.0	2.6

Figure 7. Variations found in average wet density values among 11 gages using direct transmission measurement.

of the variance. Any pairs of means with differences larger than the WSD are accepted as being different with a 5 percent probability of error. The WSD was 4.1, 4.0, 3.8, 4.0 and 3.7 pcf for sections A through E respectively; these are values obtained statistically and not by engineering deduction. Detailed examination of the data in Table 6 will reveal that of all the possible comparisons, 50 percent of the cases show no statisical difference within the WSD limits given above. These results seem good when consideration is given to the variables encountered.

Direct Transmission—This was one of the two auxiliary experiments made during the study. Ideally various depth readings would have been made, but time limitations prevented more than just 6-in. depth readings from being made.

Figure 7 shows the variations present with 11 direct transmission gages. The variations for direct transmission are from 7 to 9 pcf, which appears to be quite good. The standard deviations are from 2.0 to 3.5 pcf. Every section had a reduced standard deviation from the backscatter method; however, it should be realized that a smaller number of gage designs are included in this analysis than in the backscatter analysis. Of interest are the reductions in standard deviations in sections D and E relative to the other sections as compared to the backscatter results. A possible explanation of this may be that the direct transmission measurements are not as sensitive to particle size effects (which cause surface irregularities) as backscatter.

Note also from Figures 4 and 7 that when the average wet density values for section C are compared no appreciable decrease can be found, and thus it appears that the iron in this soil also affected the direct transmission readings. (The average wet density for each gage is found in Appendix D.) The increase in direct transmission wet density for section D cannot readily be explained except as a function of surface texture.

When the variations for each gage are examined more closely (Appendix E) it is seen that the between-site standard deviations are less than for backscatter. There is probably no single reason for this, although some possibilities are (a) the average density from the surface to a 6-in. depth may be more uniform from site to site than the top 3 in. or so (that measured by backscatter); and (b) surface irregularities, which vary from site to site, are reflected more in the backscatter readings.

The within-site variations for direct transmission are about one-half of those for backscatter—indicating better precision for the former. This increased precision as well as the smaller number of gages available for comparison make the WSD values for determining significance between average densities much smaller than for backscatter—only 1.2, 2.6, 1.5, 1.6 and 1.0 pcf for sections A through E respectively. This makes the comparison between gages (Appendix F) appear very poor from a strictly statistical analysis. If the same WSD values used for backscatter of approximately 4.0 pcf had also been used for direct transmission, the agreement between gages would have been better. If WSD values were based on engineering-deduced density values, this could be done; however, since it is entirely a statistical parameter, it cannot.

Air Gap—This was the other auxiliary experiment made during the study. Briefly, the air gap technique consists of taking a reading at some predetermined height above the surface to be tested (usually $\frac{1}{2}$ to $2\frac{1}{2}$ in.) and dividing this value by that from a surface backscatter reading. The resulting figure is called the air gap ratio. Since only two gages were prepared to make measurements by this technique and then only a single measurement was made on each site, the amount of data available for analysis was quite limited.

The average wet densities for sections A through D were 122.0, 118.0, 118.5, and 135.3 pcf. The only value obtained on section E was 147.1 pcf. These averages are quite close to the expected values on all sections. The results between the two gages were quite close on the four sections tested by each—section A, 122.6 and 121.4; section B, 117.0 and 119.0; section C, 117.6 and 119.3; and section D, 134.9 and 135.9 pcf. It is regrettable that more data could not have been obtained using this technique because it appears to be very promising.

Moisture Calibration

Calibration curves were obtained for 30 moisture gages. As is normally accepted for moisture calibration, a linear regression was used. The results of this portion of

TABLE 7

STANDARD ERRORS FOR MOISTURE CALIBRATION CURVES

Gage No.	Standard Errors (pcf)	Gage No.	Standard Errors (pcf)
1	0.7	20	0.6
2	0.7	21	0.3
3	3.6	22	0.4
4	0.7	23	0.3
5	0.5	24	0.7
6	0.7	25	0.3
7	0.9	27	0.6
8	0.5	28	3.6
9	2.1	29	3 7
10	0.7	30	0.1
11	0.2	31	0.3
12	2.6	32	1.0
13	4.4	35	0.6
14	0.6		
15	1.8	Average standard deviation	1.2
16	2.2		
19	0.3		

the analysis indicate very good accuracy, as evidenced by the predominantly small standard errors given in Table 7, This indicates rather conclusively that the total moisture present is well correlated with the thermalized neutrons counted. A typical moisture calibration curve is shown in Figure 8.

Moisture Measurements on Test Sections

The variations in moisture measurements within each test section are shown in Figure 9. The average moisture value obtained by each gage in rank order on each test section is shown in Appendix G. No valid comparison of results with actual field moisture content can be made because the moisture content during compaction was not determined and changes in moisture content may have occurred between construction and testing, and because it rained during the Correlation and Conference. The following observations, however, should be made:

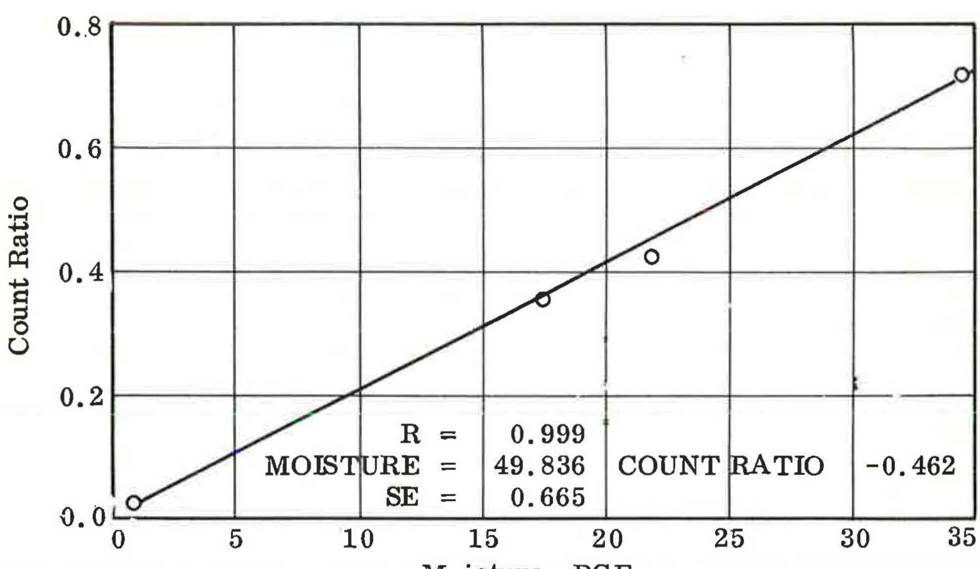
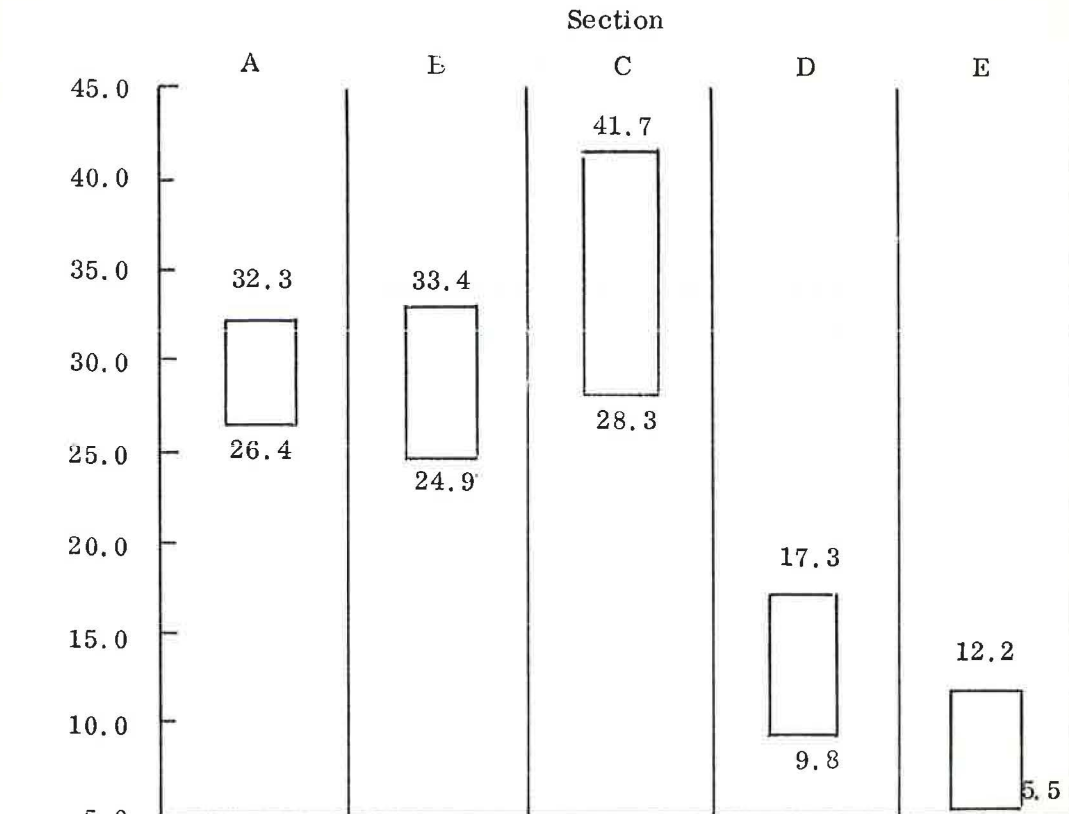


Figure 8. Typical moisture calibration curve.

TABLE 8

NUMBER OF SECTIONS ON WHICH NO STATISTICAL DIFFERENCE WAS FOUND BETWEEN GAGES (Moisture, pcf)

Gage No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		5	0	4	2	2	4	5	3	5	5	3	0	4	2	2	2	5	4	5	2	5	4	3	5	5	4	4
2			1	3	2	2	4	5	3	5	5	3	0	4	1	2	2	5	4	5	2	5	4	3	5	5	4	4
3				0	0	0	0	0	2	2	1	2	3	2	2	2	0	0	0	1	0	1	1	3	2	1	0	0
4					5	4	5	2	3	3	3	1	0	2	3	3	4	4	5	3	4	2	2	0	2	2	4	3
5						5	5	2	2	2	2	1	0	1	2	2	5	2	5	2	3	2	1	0	2	2	4	1
6							2	2	1	2	2	0	0	2	1	1	5	2	2	2	3	2	1	0	2	2	3	1
7								3	3	3	4	2	0	3	3	3	2	4	5	3	4	2	4	2	4	4	5	3
8									1	5	5	3	0	5	2	1	1	5	3	5	2	4	4	3	5	5	4	4
9										1	2	4	2	0	5	5	1	3	3	2	4	0	2	2	0	2	2	4
10											5	3	0	5	1	1	1	5	3	5	1	5	4	3	5	5	4	4
11												3	0	5	1	2	2	5	4	5	2	5	4	3	5	5	4	4
12													2	2	3	3	0	3	2	3	2	2	3	3	3	3	2	4
13														0	2	2	0	0	0	0	1	0	0	1	0	0	0	0
14															0	0	0	4	3	5	1	5	4	2	5	5	4	4
15																5	2	2	3	1	2	0	1	3	1	1	2	2
16																	2	2	3	1	2	0	1	3	2	1	2	2
20																		2	3	2	4	1	0	0	2	1	3	0
21																			5	5	2	4	3	2	5	5	4	4
22																				4	4	2	3	2	4	4	5	3
23																					2	5	4	3	5	5	4	4
24																						1	2	1	1	1	3	2
25																							3	3	5	5	3	4
27																								2	2	3	3	4
28																									3	3	2	3
30																										5	4	4
31																											3	4
32																												3



	A	B	C	D	E
Average, pcf	29.2	29.9	36.0	12.9	7.8
Optimum, pcf	23.0	24.0	29.0 (Approx)	6.0 (Approx)	4.5
Std. Deviation	1.5	1.8	2.9	2.2	2.0

Figure 9. Variations in moisture for each test section.

1. The moisture contents during construction for sections A, B, C and D were observed to be slightly higher than optimum. Section E, a pug mill mixed material, was compacted very near optimum.

2. The data in Figure 9 indicate that the average moisture results are slightly above optimum. This suggests that the moisture results reflect the amount of free moisture. (Free moisture, i. e., moisture that can be driven off at 100 C should not be confused with chemically bound moisture, which can be driven off at 300 C.) From a practical viewpoint this is fortunate since compaction control in the field is based on the amount of free moisture.

3. It should be noted, however, that the nuclear devices measure total moisture (or actually hydrogen), i. e., free and chemically bound moisture. In sections A, B and C considerable chemically bound moisture was found (see Table 4), which fortunately is not believed to be reflected in the results. A possible explanation is that the presence of some chemical such as sulfur in the calibration blocks might have biased the calibration curves.

4. The standard deviations shown in Figure 9 indicate that there are smaller differences among all moisture gages than among all density gages, a fact generally accepted. The greatest difference between gages was found in section C, which had

the highest moisture content. A possible reason for this was the high iron content (19.8%) of this soil, which may have affected some gages more than others.

5. Table 8 shows the number of sections (obtained from Appendix H) on which the moisture gages compare within statistical limits. The WSD for each section was 2.0, 2.1, 2.3, 1.6 pcf, for sections A through E respectively. The data show that of all possible comparisons, 55 percent of the gages do compare within the statistical limits.

6. The data in Appendix I show the precision of the individual moisture gages to be very good as evidenced by the within-site standard deviation being less than the between-site standard deviation on 95 percent of the sections. For every gage the average standard deviation is less for within sites than for between sites.

COMMENTS

The Correlation and Conference was the first of its type to be held. Although much valuable information was obtained from it, it should be borne in mind that some unforeseen problems were encountered.

1. The calibration standards are a vital part of the correlation. The authors spent about 6 to 8 months and exhausted all possibilities, to their knowledge, in obtaining homogeneous natural materials for density standards. Although there is a possibility that other natural materials can be found, it is believed that their homogeneity would be of the same order as the ones used. Glass blocks of different specific gravity could be an answer to this problem if they could be manufactured at the desired homogeneity. Glass manufacturers that were contacted at the time stated that this was not possible.

Moisture calibration blocks were satisfactory because they were man-made; however, their homogeneity is always a question. Since they were prepared from materials that had chemically bound moisture, evaporation was no problem. However, moisture calibration blocks prepared from materials with free moisture might produce more useful results. The authors planned to prepare a standard by using glass beads and water, but since the beads did not arrive on time, this was not done. In any case, the preparation of standards consumes a great deal of time and at least a year should be allowed for this.

2. Before the conference many persons were consulted about the spacing of devices during calibration and field testing. From these consultations it was concluded that 8 to 10 ft would be sufficient to eliminate interference. The spacing during the conference was around 12 to 15 ft. As it turned out this spacing was questioned. Some of the gages are claimed to be more sensitive than others to nearby sources. It is therefore suggested that the distance between devices be around 30 ft if possible.

3. Some of the randomizations, such as the randomization of readings on each block by each device, seem to be very time-consuming and hardly worthwhile.

4. Weather conditions seem to be the governing factor in field testing. The test road was covered with plastic material and in order to protect the road at all times it was decided to test through the plastic. This was a last-minute decision made at the first conference session and it turned out to be very valuable. It was later determined that the plastic did not affect the counts that were taken.

Though it helped, testing through the plastic material did not eliminate the rain problem completely since, toward the end of the testing, holes had to be punched for direct transmission measurements. Also, because they are airtight, large sheets of plastic are susceptible to winds. It is therefore suggested that a shelter be built over the test road.

5. During the data analysis it was found that some devices reported results consistently "out" from the others. Since there was no justification for excluding the data, some of the analyses resulted in great variability. Through much correspondence, which consumed a great deal of time, it was found that some of the devices that gave "out" data had malfunctioned. For example, one participant reported that his device would not start but if he kicked it slightly it would. It was then realized that such things should have been reported on the data sheets at the time of testing.

CONCLUSIONS

As was mentioned earlier, a great deal of information can be gleaned from the voluminous data obtained during the study. However, much of the information is of vital interest to only the particular participant involved. The following conclusions, therefore, are based only on general trends.

Wet Density Calibration

1. The large magnitudes of the standard errors found in the wet density calibration curves are believed due to the chemical composition of the standard density blocks, thus indicating the importance of chemical effect on the nuclear readings.
2. These standard errors also indicate little difference between gages as regards sensitivity to chemical effect.
3. The direct transmission standard errors are generally less than the corresponding backscatter standard errors, possibly indicating less sensitivity to chemical effects by the former technique.

Field Testing, Backscatter Technique

1. Standard deviations among gage averages when measuring wet density of the same material may be as high as 5.3 pcf.
2. The largest component of variation in testing is found to be between sites rather than within sites. This indicates that testing time can most efficiently be utilized by taking a single measurement at more sites.
3. The data indicate that there was no statistical difference between gages in 50 percent of the cases.

Field Testing, Direct Transmission Technique

1. For a limited number of measurements at a 6-in. depth, the standard deviation among gage averages when measuring wet density of the same material may be as high as 3.5 pcf.
2. A single measurement at several sites is again the most efficient use of testing time.

Field Testing, Air Gap Technique

1. This method appears very promising; however, sufficient data were not obtained to allow any generalized conclusions to be made.

Moisture Calibration

1. A very good correlation was found between total moisture and count ratio, as was evidenced by the small magnitude of standard errors found in the calibration curves.

Moisture Test Sections

1. Standard deviations among gage averages when measuring moisture of the same material may be as high as 2.9 pcf.
2. There is less difference between all moisture gage averages than between all density gage averages.
3. The data indicate that there was no statistical difference between gages in 55 percent of the cases.
4. Moisture measurements are very precise, which indicates that single measurements at each site should suffice.

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Finally, sincere thanks go to all the participants, without whose interest and excellent cooperation the Correlation and Conference could not have been accomplished. Their constructive criticism of the rough draft is also appreciated. It is believed that the incorporation of their comments, where applicable, added to the value of this report.

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Appendix A

MEANS OF WET DENSITY MEASUREMENTS BY BACKSCATTER TECHNIQUE, pcf

Section A		Section B		Section C		Section D		Section E	
Gage #	W. D.	Gage #	W. D.	Gage #	W. D.	Gage #	W. D.	Gage #	W. D.
10	134.4	10	131.4	10	132.1	35	146.0	10	158.1
25	133.8	25	130.2	25	131.1	10	142.1	2	154.5
23	133.7	2	127.7	32	130.1	4	141.1	8	153.1
2	130.4	4	126.7	8	129.5	31	140.9	31	152.9
8	129.4	31	126.0	2	129.2	25	140.4	35	152.6
31	129.2	13	125.6	1	128.0	8	139.4	1	152.4
4	128.5	8	125.6	5	128.0	2	138.9	4	151.1
21	128.5	9	125.3	6	127.3	1	138.5	23	150.8
13	128.4	23	125.2	24	127.3	13	138.0	21	150.1
35	128.0	14	124.5	30	126.9	23	137.1	25	150.0
30	127.0	1	124.5	7	126.7	9	135.9	30	149.1
1	126.8	5	124.4	13	126.7	14	135.4	13	149.0
24	125.7	21	124.4	23	126.5	21	135.2	5	147.9
14	125.6	24	123.9	27	125.9	30	135.0	15	147.7
15	125.5	15	123.5	4	125.6	15	134.8	28	147.5
5	125.4	30	122.6	20	125.4	16	133.6	9	147.2
9	125.3	16	122.0	28	125.1	28	133.2	14	147.2
32	125.0	32	121.9	21	124.8	22	132.5	16	147.2
28	124.6	28	121.7	14	124.7	5	131.9	3	145.7
16	124.5	12	121.5	31	124.7	24	131.1	22	145.6
6	124.2	35	121.4	11	124.4	12	130.9	24	145.0
11	123.2	11	121.2	15	123.6	3	130.3	12	144.1
12	122.2	6	120.1	16	123.2	11	130.0	11	143.5
27	122.2	20	119.4	9	122.2	32	129.2	6	143.2
7	121.7	7	118.4	35	120.5	20	125.9	32	143.0
3	121.4	3	117.5	12	120.1	18	125.4	18	142.7
20	121.2	18	117.4	3	119.7	7	124.8	7	142.3
18	121.1	27	116.3	18	114.2	27	124.5	20	140.8
								27	140.6
$\bar{X}$	126.3	$\bar{X}$	123.3	$\bar{X}$	125.6	$\bar{X}$	133.8	$\bar{X}$	147.6
σ	3.8	σ	3.6	σ	3.7	σ	5.3	σ	4.5

Appendix B

SOURCES OF VARIATION IN WET DENSITY BY BACK-SCATTER TECHNIQUE, STANDARD DEVIATION, pcf

Gage 1			Gage 2			Gage 3		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	3.03	1.34	A	3.86	.25	A	3.05	.50
B	3.52	.94	B	2.87	.48	B	3.84	.47
C	2.78	.51	C	3.63	.47	C	2.78	.93
D	2.36	.50	D	3.71	.40	D	5.65	.57
E	3.71	.58	E	3.09	.17	E	5.46	.49
Avg.	3.12	.84	Avg.	3.45	.37	Avg.	4.32	.62

Gage 4			Gage 5			Gage 6		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	2.45	.55	A	2.87	.30	A	5.49	.88
B	1.93	.93	B	1.70	.44	B	3.21	.42
C	2.39	.40	C	1.46	.39	C	3.43	.84
D	3.17	.42	D	2.87	.76	D	Mal-function	--
E	3.32	.54	E	4.37	.32	E	5.14	.69
Avg.	2.79	.62	Avg.	2.85	.47	Avg.	4.44	.74

Gage 7			Gage 8			Gage 9		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	6.14	1.41	A	4.15	1.07	A	3.90	.46
B	3.22	1.10	B	3.87	.94	B	4.37	.42
C	4.56	.99	C	1.33	.64	C	2.54	.57
D	5.29	1.04	D	2.62	.99	D	4.14	.41
E	4.57	1.07	E	4.87	1.19	E	6.47	.57
Avg.	4.85	1.13	Avg.	3.60	.98	Avg.	4.47	.49

Appendix B (Continued)

Gage 10			Gage 11			Gage 12		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	4.30	1.57	A	5.86	.76	A	4.30	.44
B	3.07	.96	B	4.67	.61	B	2.49	.45
C	1.60	1.20	C	3.75	1.07	C	2.14	.28
D	2.47	1.07	D	5.23	.59	D	4.38	.40
E	5.04	.95	E	5.34	.88	E	8.06	.68
Avg.	3.52	1.17	Avg.	5.02	.81	Avg.	4.76	.47

Gage 13			Gage 14			Gage 15		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	3.59	.36	A	4.20	1.33	A	3.70	.48
B	2.35	.47	B	2.75	1.61	B	3.12	.59
C	1.87	.47	C	1.78	.87	C	1.22	.32
D	4.02	.47	D	5.17	1.02	D	3.80	.47
E	3.94	.30	E	4.96	1.14	E	5.31	.53
Avg.	3.28	.42	Avg.	3.94	1.22	Avg.	3.67	.49

Gage 16			Gage 18			Gage 20		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	3.68	.42	A	6.22	.86	A	7.89	.94
B	2.29	.67	B	4.60	.63	B	3.61	.39
C	6.30	.79	C	3.46	.48	C	2.93	.50
D	5.43	.46	D	6.88	.55	D	6.37	1.65
E	4.66	.36	E	5.78	.55	E	4.75	.64
Avg.	3.85	.56	Avg.	5.52	.62	Avg.	5.42	.94

Appendix B (Continued)

Gage 21			Gage 22			Gage 23		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	1.91	1.14	A	Mal-function	--	A	3.90	.73
B	4.23	.88	B	"	"	B	2.92	.90
C	2.85	1.54	C	"	"	C	2.34	.79
D	3.79	1.59	D	3.36	.84	D	4.36	.48
E	3.45	1.41	E	4.29	.99	E	4.49	.84
Avg.	3.35	1.34	Avg.	3.82	.92	Avg.	3.70	.76

Gage 24			Gage 25			Gage 27		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	1.83	.64	A	2.56	.71	A	6.75	.75
B	2.56	.36	B	2.06	.59	B	3.94	1.12
C	1.51	.50	C	2.30	1.20	C	4.14	1.03
D	2.99	.40	D	3.84	.85	D	4.07	1.03
E	3.40	.32	E	6.91	1.23	E	4.70	.82
Avg.	2.56	.46	Avg.	3.96	.95	Avg.	4.84	.96

Gage 28			Gage 30			Gage 31		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	3.37	.37	A	4.96	.98	A	2.13	.83
B	2.41	.35	B	3.51	1.58	B	2.23	.59
C	1.89	.22	C	2.66	.97	C	1.58	.51
D	3.46	.48	D	4.00	.92	E	2.11	.28
E	3.87	.28	E	3.54	.97	E	4.07	.33
Avg.	3.09	.35	Avg.	3.81	1.12	Avg.	2.57	.55

Gage 32		
Section	Between Site	Within Site
A	5.26	.80
B	4.54	.85
C	3.11	.94
D	5.22	1.07
E	5.72	.91
Avg.	4.86	.92

INSTANCES OF NO SIGNIFICANT DIFFERENCES BETWEEN MEANS OF
WET DENSITY MEASUREMENTS BY BACK-SCATTER TECHNIQUE

(SECTION A: Means less than WSD* of 4.1, pcf = 56%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	23	24	25	27	28	30	31	32	35
1		X		X	X	X		X	X		X		X	X	X	X			X		X			X	X	X	X	X
2				X				X		X			X						X	X		X			X	X		X
3					X	X	X		X		X	X				X	X	X					X	X			X	
4					X			X	X				X	X	X	X			X		X			X	X	X	X	X
5						X	X	X	X		X	X	X	X	X	X			X		X		X	X	X	X	X	X
6							X	X			X	X		X	X	X	X	X			X		X	X	X		X	X
7									X		X	X		X	X	X	X	X			X		X	X			X	
8													X	X	X				X		X				X	X		X
9											X	X	X	X	X	X			X		X		X	X	X	X	X	X
10																				X		X						
11												X		X	X	X	X	X			X		X	X	X		X	
12														X	X	X	X	X			X		X	X			X	
13														X	X	X			X		X			X	X	X	X	X
14															X	X			X		X		X	X	X	X	X	X
15																X			X		X		X	X	X	X	X	X
16																	X	X	X		X		X	X	X		X	X
18																		X					X	X			X	
20																								X			X	
21																					X			X	X	X	X	X
23																							X					
24																							X	X	X	X	X	X
25																												
27																								X			X	
28																									X		X	X
30																										X	X	X
31																												X
32																												X

* Based on number and precision of gage at 95% confidence.

Appendix C

(SECTION B: Means less than WSD of 4.0, pcf = 57%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	23	24	25	27	28	30	31	32	35
1		X		X	X			X	X		X	X	X	X	X	X			X	X	X			X	X	X	X	X
2				X	X			X	X	X			X	X					X	X	X	X				X		
3						X	X				X						X	X					X					X
4					X			X	X				X	X	X				X	X	X	X				X		
5								X	X		X	X	X	X	X	X			X	X	X			X	X	X	X	X
6							X				X	X			X	X	X	X			X		X	X	X		X	X
7											X	X				X	X	X					X	X			X	X
8								X					X	X	X	X			X	X	X			X	X	X	X	
9												X	X	X	X	X			X	X	X			X	X	X	X	X
10																						X						
11												X		X	X	X	X	X	X		X			X	X		X	X
12														X	X	X		X	X	X	X			X	X		X	X
13														X	X	X			X	X	X			X	X	X	X	
14															X	X			X	X	X			X	X	X	X	X
15																X			X	X	X			X	X	X	X	X
16																		X	X	X	X			X	X		X	X
18																		X					X	X				
20																							X		X		X	X
21																				X	X			X	X	X	X	X
23																					X			X	X	X	X	X
24																								X	X	X	X	X
25																												
27																												
28																									X		X	X
30																										X	X	X
31																												
32																												X

(SECTION C: Means less than WSD of 3.8, pcf = 58%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	23	24	25	27	28	30	31	32	35
1		X		X	X	X	X	X			X		X	X				X	X	X	X	X	X	X	X	X	X	
2				X	X	X	X	X		X			X							X	X	X	X		X		X	
3									X			X				X												X
4					X	X	X		X		X		X	X	X	X		X	X	X	X		X	X	X	X		
5						X	X	X			X		X	X				X	X	X	X	X	X	X	X	X	X	
6							X	X			X		X	X	X			X	X	X	X		X	X	X	X	X	
7								X			X		X	X	X	X		X	X	X	X		X	X	X	X	X	
8										X			X							X	X	X	X		X		X	
9											X	X		X	X	X		X	X				X	X		X		X
10																						X					X	
11													X	X	X	X		X	X	X	X		X	X	X	X		
12															X	X												X
13													X	X	X			X	X	X	X		X	X	X	X	X	
14															X	X		X	X	X	X		X	X	X	X		
15																X		X	X	X	X		X	X	X	X		X
16																		X	X	X			X	X	X	X		X
18																												
20																			X	X	X		X	X	X	X		
21																				X	X		X	X	X	X		
23																					X		X	X	X	X	X	
24																							X	X	X	X	X	
25																											X	
27																								X	X	X		
28																									X	X		
30																										X	X	
31																												
32																												

(SECTION D: Means less than WSD of 4.0, pcf = 34%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X				X	X	X			X	X	X				X		X		X			X	X		
2				X				X	X	X			X	X					X		X		X			X	X		
3					X						X	X				X				X		X			X			X	
4								X		X			X										X				X		
5											X	X		X	X	X		X		X		X			X	X		X	
7																	X	X						X					
8									X	X			X								X		X				X		
9													X	X	X	X			X	X	X				X	X			
10																							X				X		X
11												X				X				X		X			X			X	
12															X	X				X		X			X			X	
13														X	X				X		X		X			X	X		
14															X	X			X	X	X				X	X			
15																X			X	X	X	X			X	X			
16																			X	X	X	X			X	X			
18																		X						X				X	
20																								X				X	
21																				X	X				X	X			
22																						X			X	X		X	
23																							X		X	X	X		
24																									X	X		X	
25																											X		
27																													
28																										X			
30																													
31																													
32																													

(SECTION E: Means less than WSD of 3.7, pcf = 42%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X				X					X						X		X		X			X	X		X
2				X				X		X																	X		X
3					X	X	X		X		X	X	X	X	X	X	X			X		X			X	X		X	
4					X			X					X		X				X		X		X		X	X	X		X
5									X				X	X	X	X			X	X	X	X	X		X	X			
6							X				X	X					X	X		X		X		X				X	
7											X	X					X	X		X		X		X				X	
8																			X		X		X				X		X
9												X	X	X	X	X			X	X	X	X	X		X	X			
10																													
11												X		X		X	X	X		X		X		X				X	
12														X		X	X	X		X		X		X	X			X	
13														X	X	X			X	X	X		X		X	X			X
14															X	X			X	X	X	X	X		X	X			
15																X			X	X	X	X	X		X	X			
16																			X	X	X	X	X		X	X			
18																		X		X		X		X				X	
20																								X				X	
21																					X		X		X	X	X		X
22																						X			X	X		X	
23																							X		X	X	X		X
24																												X	
25																									X	X	X		X
27																												X	
28																													
30																													X
31																													X
32																													

Appendix D

MEANS OF WET DENSITY MEASUREMENT BY DIRECT TRANSMISSION TECHNIQUE, pcf

Section A		Section B		Section C		Section D		Section E	
Gage #	W. D.	Gage #	W. D.	Gage #	W. D.	Gage #	W. D.	Gage #	W. D.
19	134.3	28	129.3	22	128.7	19	147.2	18	153.4
28	132.3	6	126.4	28	128.6	18	144.2	29	153.4
6	130.4	22	124.7	6	125.0	13	144.1	15	152.8
22	129.8	19	123.2	18	122.7	29	143.7	13	152.6
13	129.3	13	122.6	13	122.6	15	143.1	22	151.4
3	129.1	18	122.3	29	122.0	6	142.8	16	151.3
29	128.3	29	122.0	3	121.5	3	142.0	19	151.2
18	128.0	3	121.9	15	120.7	16	141.9	6	151.0
15	127.9	15	121.8	16	120.4	9	140.2	3	150.8
16	127.2	16	121.3	19	120.2	22	139.9	9	149.5
9	125.1	9	118.5	9	117.6				
$\overline{X}$	129.2	$\overline{X}$	123.1	$\overline{X}$	122.7	$\overline{X}$	142.9	$\overline{X}$	151.1
σ	2.7	σ	2.9	σ	3.5	σ	2.0	σ	2.6

Appendix E

SOURCES OF VARIATION IN WET DENSITY BY DIRECT TRANSMISSION TECHNIQUE
STANDARD DEVIATION, pcf

Gage 3			Gage 9			Gage 13		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	5.22	.26	A	2.16	.28	A	2.61	.25
B	2.92	1.57	B	2.66	.30	B	3.60	.33
C	1.25	.33	C	2.31	.32	C	2.91	.30
D	1.25	.85	D	1.50	.26	D	1.93	.17
E	2.33	.26	E	2.73	.14	E	1.92	.20
Avg.	3.20	.83	Avg.	2.31	.26	Avg.	2.67	.26

Gage 15			Gage 16			Gage 18		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	2.17	.28	A	2.14	.22	A	1.88	.31
B	2.14	.20	B	2.26	.24	B	2.73	.48
C	1.34	.22	C	1.61	.28	C	1.94	.35
D	1.54	.40	D	1.49	.20	D	1.00	.22
E	2.40	.20	E	2.42	.20	E	2.28	.24
Avg.	1.95	.26	Avg.	2.02	.22	Avg.	2.05	.33

Gage 19			Gage 22			Gage 28		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	3.45	.26	A	1.36	.41	A	.89	.10
B	4.27	.20	B	3.32	.39	B	1.34	.10
C	2.05	.39	C	3.09	.40	C	1.33	.10
D	.85	.20	D	2.82	.24	D	-	-
E	3.37	.22	E	4.54	.14	E	1.57	.00
Avg.	3.04	.26	Avg.	3.68	.33	Avg.	1.31	.10

Gage 29		
Section	Between Site	Within Site
A	4.96	.17
B	4.27	.22
C	4.02	.41
D	2.48	.28
E	2.27	.37
Avg.	3.75	.30

Appendix F

INSTANCES OF NO SIGNIFICANT DIFFERENCES BETWEEN MEANS OF WET DENSITY MEASUREMENTS BY BACK-SCATTER TECHNIQUE

(SECTION A: Means less than WSD of 1.2, pcf = 23%)

Gage #	3	6	9	13	15	16	18	19	22	28	29
3				X					X		X
6				X					X		
9											
13									X		X
15						X	X				X
16							X				X
18											X
19											
22											
28											

(SECTION B: Means less than WSD of 2.6, pcf = 45%)

Gage #	3	6	9	13	15	16	18	19	22	28	29
3				X	X	X	X	X			X
6									X		
9											
13					X	X	X	X	X		X
15						X	X	X			X
16							X	X			X
18								X	X		X
19									X		X
22											
28											

(SECTION C: Means less than WSD of 1.5, pcf = 25%)

Gage #	3	6	9	13	15	16	18	19	22	28	29
3				X	X	X	X	X			X
6											
9											
13							X				X
15						X		X			X
16								X			
18											X
19											
22										X	
28											

Appendix F (Continued)

(SECTION D: Means less than WSD of 1.6, pcf = 29%)

Gage #	3	6	9	13	15	16	18	19	22	29
3		X			X	X				
6				X	X	X	X			X
9									X	
13					X		X			X
15						X	X			X
16										
18										X
19										
22										

(SECTION E: Means less than WSD of 1.0, pcf = 29%)

Gage #	3	6	9	13	15	16	18	19	22	29
3		X				X		X	X	
6						X		X	X	
9										
13					X		X			X
15							X			X
16								X	X	
18										X
19									X	
22										
28										

Appendix G

MEANS OF MOISTURE MEASUREMENT, pcf

Section A		Section B		Section C		Section D		Section E	
Gage #	w	Gage	w	Gage	w	Gage	w	Gage	w
6	32.3	6	33.4	6	41.7	28	17.3	28	12.2
20	32.3	5	32.7	20	41.1	3	17.1	15	11.6
4	31.5	20	32.7	32	39.9	13	16.4	16	11.4
5	30.9	24	31.7	5	39.4	15	16.4	3	11.2
16	30.9	15	31.6	24	39.0	16	16.3	9	10.6
15	30.8	4	31.5	7	38.4	9	15.6	13	10.4
24	30.5	9	31.4	22	38.0	12	15.2	12	9.7
9	30.3	16	31.3	27	37.4	35	14.4	24	9.5
7	30.0	7	31.1	4	37.2	24	12.6	5	7.7
22	29.9	22	30.7	9	36.8	4	12.1	20	7.6
32	29.6	21	29.9	16	36.5	11	12.1	32	7.0
12	29.4	35	29.8	15	36.2	2	11.9	7	7.0
8	28.9	1	29.6	21	36.1	5	11.9	6	6.9
28	28.9	32	29.6	12	35.6	6	11.9	31	6.8
2	28.8	2	29.4	1	35.5	21	11.9	23	6.8
1	28.8	28	29.4	2	35.5	31	11.9	21	6.8
23	28.8	11	29.3	35	35.5	23	11.8	22	6.8
27	28.8	30	29.3	11	35.5	22	11.8	2	6.6
11	28.6	27	29.2	23	35.4	30	11.7	25	6.6
35	28.6	31	29.2	10	35.3	20	11.7	4	6.5
21	28.5	12	29.1	31	34.8	1	11.6	10	6.4
31	28.4	23	29.0	8	34.4	7	11.6	1	6.4
10	28.2	10	28.6	30	34.4	10	11.5	8	6.4
30	28.2	14	28.5	25	33.7	25	11.4	30	6.4
14	28.1	8	28.1	28	33.4	32	11.4	11	6.0
25	27.3	25	27.9	14	33.3	14	11.2	35	6.0
3	26.4	3	27.4	3	29.9	8	11.1	14	5.9
13	24.9	13	24.9	13	28.3	27	9.8	27	5.5
$\bar{X}$	29.2	$\bar{X}$	29.9	$\bar{X}$	36.0	$\bar{X}$	12.9	$\bar{X}$	7.8
σ	1.5	σ	1.8	σ	2.9	σ	2.2	σ	2.0

Appendix H

INSTANCES OF NO SIGNIFICANT DIFFERENCES BETWEEN MEANS OF MOISTURE MEASUREMENTS

(SECTION A: Means less than WSD of 2.0, pcf = 58%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		X					X	X	X	X	X	X		X				X	X	X	X	X	X	X	X	X	X	X
2							X	X	X	X	X	X		X				X	X	X	X	X	X	X	X	X	X	X
3										X				X								X			X			
4					X	X	X		X						X	X	X		X		X						X	
5						X	X		X			X			X	X	X		X		X						X	
6																X	X				X							
7								X	X	X	X	X		X	X	X		X	X	X	X		X	X	X	X	X	X
8									X	X	X	X		X	X			X	X	X	X	X	X	X	X	X	X	X
9											X	X			X	X		X	X	X	X		X	X		X	X	X
10											X	X		X				X	X	X		X	X	X	X	X	X	X
11												X		X				X	X	X	X	X	X	X	X	X	X	X
12														X	X	X		X	X	X	X		X	X	X	X	X	X
13																												
14																		X	X	X		X	X		X	X	X	X
15																X	X		X		X			X			X	
16																	X		X		X						X	
20																					X							
21																			X	X		X	X	X	X	X	X	X
22																				X	X		X	X	X	X	X	X
23																					X	X	X	X	X	X	X	X
24																							X	X			X	X
25																							X	X	X	X		X
27																								X	X	X	X	X
28																									X	X	X	X
30																										X	X	X
31																											X	X
32																												X

(SECTION B: Means less than WSD of 2.1, pcf = 58%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X			X	X	X	X	X	X		X	X	X		X	X	X		X	X	X		X	X	X
2			X				X	X	X	X	X	X		X		X		X	X	X		X	X	X	X	X	X	X
3										X	X	X		X						X			X	X	X	X		
4					X	X	X		X						X	X	X	X	X		X						X	X
5						X	X		X						X	X	X		X		X							
6									X						X		X				X							
7									X		X	X			X	X	X	X	X		X		X	X	X	X	X	X
8										X	X	X		X				X		X			X	X	X	X	X	X
9															X	X	X	X	X		X			X			X	X
10											X	X		X				X		X		X	X	X	X	X	X	X
11												X		X		X		X	X	X		X	X	X	X	X	X	X
12														X				X	X	X		X	X	X	X	X	X	X
13																												
14																		X		X		X	X	X	X	X	X	X
15																X	X	X	X		X						X	X
16																	X	X	X		X			X	X		X	X
20																			X		X							
21																			X	X	X	X	X	X	X	X	X	X
22																				X	X		X	X	X	X	X	X
23																						X	X	X	X	X	X	X
24																												X
25																							X	X	X	X	X	X
27																								X	X	X	X	X
28																									X	X	X	X
30																										X	X	X
31																											X	X
32																												X

(SECTION C: Means less than WSD of 2.3, pcf = 46%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X				X	X	X	X	X			X	X		X		X		X	X	X	X	X		X
2				X				X	X	X	X	X			X	X		X		X		X	X	X	X	X		X
3													X															
4					X		X		X	X	X	X			X	X		X	X	X	X		X					X
5						X	X										X		X				X				X	
6																	X										X	
7									X						X	X			X		X		X				X	
8										X	X	X		X	X	X		X		X		X		X	X	X		X
9										X	X	X			X	X		X	X	X	X		X			X		X
10											X	X		X	X	X		X		X		X	X	X	X	X		X
11												X		X	X	X		X		X		X	X	X	X	X		X
12															X	X		X		X		X	X	X	X	X		X
13																												
14																				X		X		X	X	X		X
15																X		X	X	X			X		X	X		X
16																		X	X	X			X		X	X		X
20																					X						X	
21																			X	X			X		X	X		X
22																					X		X				X	
23																						X	X	X	X	X		X
24																							X				X	
25																								X	X	X		X
27																												X
28																									X	X		X
30																										X		X
31																												X
32																												

(SECTION D: Means less than WSD of 1.6, pcf = 49%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X	X	X	X	X		X	X			X			X	X	X	X	X	X			X	X	X	
2				X	X	X	X	X		X	X			X			X	X	X	X	X	X			X	X	X	
3									X				X		X	X								X				
4					X	X	X	X		X	X			X			X	X	X	X	X	X			X	X	X	
5						X	X	X		X	X			X			X	X	X	X	X	X			X	X	X	
6							X	X		X	X			X			X	X	X	X	X	X			X	X	X	
7								X		X	X			X				X	X	X	X	X			X	X	X	
8										X	X			X				X	X	X	X	X	X		X	X	X	
9												X	X		X	X												X
10											X			X				X	X	X	X	X			X	X	X	
11														X			X	X	X	X	X	X			X	X	X	
12													X		X	X												X
13															X	X								X				
14																		X	X	X	X	X	X		X	X	X	
15																X								X				
16																								X				
20																		X	X	X	X				X	X		
21																			X	X	X	X			X	X	X	
22																				X	X	X			X	X	X	
23																					X	X			X	X	X	
24																						X			X	X	X	
25																									X	X	X	
27																												
28																												
30																										X	X	
31																												
32																												

(SECTION E: Means less than WSD of 1.6, pcf = 54%)

Gage #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	20	21	22	23	24	25	27	28	30	31	32	35
1		X		X	X	X	X	X		X	X			X			X	X	X	X		X	X		X	X	X	X
2				X	X	X	X	X		X	X			X			X	X	X	X		X	X		X	X	X	X
3									X			X	X		X	X								X				
4					X	X	X	X		X	X			X			X	X	X	X		X	X		X	X	X	X
5						X	X	X		X	X						X	X	X	X		X			X	X	X	
6							X	X		X	X			X			X	X	X	X		X	X		X	X	X	X
7								X		X	X			X			X	X	X	X		X	X		X	X	X	X
8										X	X			X			X	X	X	X		X	X		X	X	X	X
9												X	X		X	X					X							
10											X			X			X	X	X	X		X	X		X	X	X	X
11														X			X	X	X	X		X	X		X	X	X	X
12													X								X							
13															X	X					X							
14																		X	X	X		X	X		X	X	X	X
15																X								X				
16																								X				
20																		X	X	X		X			X	X	X	
21																			X	X		X	X		X	X	X	X
22																				X		X	X		X	X	X	X
23																						X	X		X	X	X	X
24																												
25																							X		X	X	X	X
27																									X	X	X	X
28																												
30																										X	X	X
31																											X	X
32																												X

Appendix I

SOURCES OF VARIATION IN MOISTURE, STANDARD DEVIATION, pcf

Gage 1			Gage 2			Gage 3		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	.93	.19	A	.55	.13	A	.94	.61
B	1.52	.24	B	1.98	.13	B	1.00	.49
C	.66	.35	C	.68	.13	C	0.96	.55
D	.31	.23	D	.27	.12	D	1.36	.49
E	1.06	.08	E	.93	.04	E	1.62	.50
Avg.	.90	.22	Avg.	.88	.11	Avg.	1.18	.53

Gage 4			Gage 5			Gage 6		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	1.01	.33	A	1.88	.22	A	1.68	.68
B	1.92	.86	B	1.34	.21	B	1.29	.35
C	.58	.48	C	.70	.26	C	1.06	.49
D	.28	.22	D	.54	.34	D	.46	.31
E	.87	.14	E	1.14	.22	E	.87	.22
Avg.	.93	.40	Avg.	1.12	.25	Avg.	1.07	.41

Gage 7			Gage 8			Gage 9		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	1.35	.38	A	.82	.22	A	1.31	.31
B	1.64	.46	B	1.84	.28	B	1.41	.42
C	1.03	.44	C	.70	.84	C	1.10	.58
D	.32	.27	D	.50	.13	D	.85	.58
E	.79	.21	E	.84	.49	E	1.39	.59
Avg.	1.03	.35	Avg.	.94	.39	Avg.	1.21	.50

Appendix I (Continued)

Gage 10		
Section	Between Site	Within Site
A	.98	.25
B	1.92	.33
C	.77	.31
D	.39	.13
E	.98	.10
Avg.	1.01	.22

Gage 11		
Section	Between Site	Within Site
A	.89	.89
B	1.42	.60
C	.80	.57
D	.52	.22
E	.53	.48
Avg.	.83	.55

Gage 12		
Section	Between Site	Within Site
A	1.11	.21
B	1.53	.22
C	.96	.24
D	.86	.27
E	1.25	.32
Avg.	1.14	.25

Gage 13		
Section	Between Site	Within Site
A	.93	.49
B	1.20	.53
C	1.03	.76
D	.93	.93
E	1.41	.51
Avg.	1.10	.64

Gage 14		
Section	Between Site	Within Site
A	.82	.89
B	.99	.45
C	.92	.57
D	.31	.25
E	.76	.30
Avg.	.76	.49

Gage 15		
Section	Between Site	Within Site
A	1.12	.34
B	.95	.83
C	.40	.81
D	.76	.51
E	1.22	.60
Avg.	.89	.62

Gage 16		
Section	Between Site	Within Site
A	1.41	.62
B	1.08	.63
C	1.31	.34
D	.68	.35
E	1.33	.22
Avg.	1.16	.43

Gage 20		
Section	Between Site	Within Site
A	.96	.65
B	.87	.55
C	.68	.40
D	.62	.20
E	.68	.23
Avg.	.76	.41

Gage 21		
Section	Between Site	Within Site
A	.79	.21
B	1.60	.42
C	.72	.30
D	.34	.16
E	.99	.21
Avg.	.89	.26

Appendix 1 (Continued)

Gage 22			Gage 23			Gage 24		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	1.18	.33	A	.90	.22	A	1.07	.24
B	1.21	.30	B	1.17	.25	B	1.19	.40
C	.94	.20	C	.53	.32	C	1.08	.20
D	.34	.16	D	.48	.17	D	.43	.19
E	.78	.10	E	.99	.14	E	1.82	1.05
Avg.	.89	.22	Avg.	.81	.22	Avg.	1.12	.42

Gage 25			Gage 27			Gage 28		
Section	Between Site	Within Site	Section	Between Site	Within Site	Section	Between Site	Within Site
A	.60	.13	A	.79	.36	A	1.03	.31
B	.91	.15	B	1.35	.54	B	1.21	.27
C	.60	.26	C	.98	.48	C	.72	.45
D	.47	.12	D	.36	.33	D	.75	.38
E	.81	.06	E	.56	.22	E	1.71	.39
Avg.	.68	.14	Avg.	.81	.39	Avg.	1.08	.36

Gage 30		
Section	Between Site	Within Site
A	.46	.57
B	1.11	.48
C	1.78	.61
D	.61	.44
E	.74	.30
Avg.	.94	.48